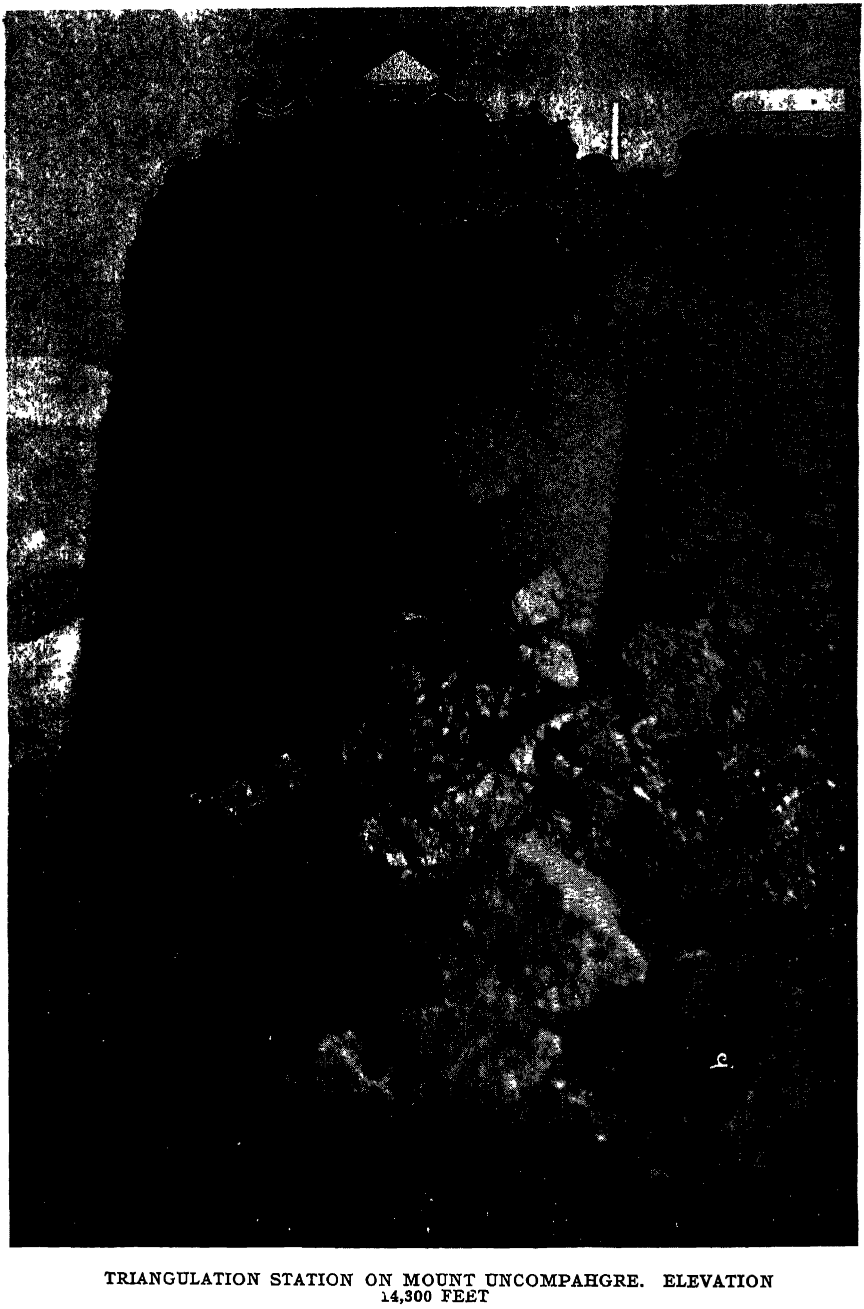

TRIANGULATION STATION ON MOUNT UNCOMPAHGRE. ELEVATION 14,300 FEET

Courtesy of U S Coast and Geodetic Survey, Washington, D C.

Cyclopedia
of
Civil Engineering

A General Reference Work on

SURVEYING, HIGHWAY CONSTRUCTION, RAILROAD ENGINEERING, EARTHWORK,
STEEL CONSTRUCTION, SPECIFICATIONS, CONTRACTS, BRIDGE ENGINEERING,
MASONRY AND REINFORCED CONCRETE, MUNICIPAL ENGINEERING,
HYDRAULIC ENGINEERING, RIVER AND HARBOR IMPROVEMENT,
IRRIGATION ENGINEERING, COST ANALYSIS, ETC.

Prepared by a Corps of

CIVIL AND CONSULTING ENGINEERS AND TECHNICAL EXPERTS OF THE
HIGHEST PROFESSIONAL STANDING

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NINE VOLUMES

CHICAGO
AMERICAN TECHNICAL SOCIETY

V. 1

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o and drawn to an exaggerated scale. The area *abcde* representing **to scale, the area ABCDE. It may** be desirable to set up the **table at some other point, as for** instance one of the corners of the **field, and run out some of the** lines to the other corners as a check **upon the work.**

Traversing, or the Method of Progression. This method is **practically the same as the** method of surveying a series of lines **with the transit, but requires** that all of the points be accessible. **It is the best method of working** as it provides a complete check **upon the survey.**

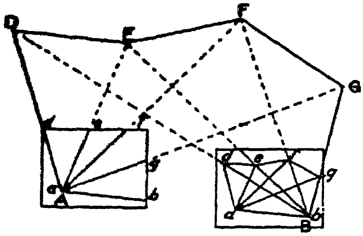

Fig. 127.

Let ABCDE, Fig. 126, be the series of lines to be surveyed by *traversing*. Set up the table at B, the second angle of the line. so that the point *b* upon the paper will be directly over the point B upon the ground. (The point *b* should be so chosen as to leave room upon the paper for as much of the traverse as possible.) Stick a needle at the point *b* and place the edge of the alidade against it. Swing the alidade around until the line of sight covers the point A. Measure BA and lay it off to the proper scale as *ba*. Now turn the alidade around the point *b* and sight to and measure the distance BC and plot it to scale as *bc*. Remove the instrument to *c* with the point *c* upon the paper directly over C upon the ground, and *cb* in the direction of CB. This is difficult to accomplish with the plane-table, but if the plot is drawn to a large scale, it must be done. If the plot is drawn to a small scale, it will be sufficiently accurate to set the table over the point C as nearly as possible in the proper direction and then turn the *board* in azimuth until *b* is in the direction of B. Stick a needle at *c* and check the length of *cb*. Swing the alidade around *c* until the line of sight covers D, measure CD and plot *cd*. Remove to D and proceed as before and so on through the traverse.

If the survey is of a closed field, the accuracy of the work will be checked by the closure of the survey.

The method of progression is especially adapted to the survey of a road, the banks of a river, etc., and often many of the details may be sketched in with the eye.

When the paper is filled, put on a new sheet, and on it, fix two points, such as D and E, which were on the former sheet and from them proceed as before. The sheets can afterward be united so that all points on both shall be in their true relative positions.

Method of Intersection. This is the most rapid method of using the plane-table. Set up the instrument at any convenient point, as A in Fig. 127 and sight to all the desired points as D, E, F, etc., which are visible, and draw indefinite lines in their directions. Measure any line as AB, B being one of the points sighted to, and plot the length of this line upon the paper to any convenient scale. Move the instrument to B so that *b* upon the paper will be directly over B upon the ground, and so that *ba* upon the paper will be in the direction of BA upon the ground as explained under the method of progression. Stick a needle at the point *b* and swing the alidade around it, sighting to all the former points in succession, and draw lines in their direction. The intersection of these two sets of lines to the several points will determine the position of the points. Connect the points as *d*, *e*, *f*, *g*, in the figure. In surveying a field, one side may be taken as the base line. In choosing the base line, care must be exercised to avoid very acute or obtuse angles: 30° and 150° being the extreme limits. The impossibility of always doing this, sometimes renders this method deficient in precision.

TOPOGRAPHICAL SURVEYING.

A topographical map is one showing the configuration of the surface of the ground of the area to be mapped and includes lakes, rivers, and all other natural features, and sometimes artificial features as well.

A topographical survey is one conducted for the purpose of acquiring information necessary for the production of a topographical map of the area surveyed.

Nearly all engineering enterprises involve a topographical survey more or less extended, depending upon the nature and

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Foreword

OF all the works of man in the various branches of engineering, none are so wonderful, so majestic, so awe-inspiring as the works of the Civil Engineer. It is the Civil Engineer who throws a great bridge across the yawning chasm which seemingly forms an impassable obstacle to further progress. He designs and builds the skeletons of steel to dizzy heights, for the architect to cover and adorn. He burrows through a great mountain and reaches the other side within a fraction of an inch of the spot located by the original survey. He scales mountain peaks, or traverses dry river beds, surveying and plotting hitherto unknown, or at least unsurveyed, regions. He builds our Panama Canals, our Arrow Rock and Roosevelt Dams, our water-works, filtration plants, and practically all of our great public works.

The importance of all of these immense engineering projects and the need for a clear, non-technical presentation of the theoretical and practical developments of the broad field of Civil Engineering has led the publishers to compile this great reference work. It has been their aim to fulfill the demands of the trained engineer for authoritative material which will solve the problems in his own and allied lines in Civil Engineering, as well as to satisfy the desires of the self-taught practical man who attempts to keep up with modern engineering developments

¶ Books on the several divisions of Civil Engineering ar many and valuable, but their information is too voluminous t be of the greatest value for ready reference. The Cyclopedia o Civil Engineering offers more condensed and less technica treatments of these same subjects from which all unnecessar duplication has been eliminated; when compiled into nin handy volumes, with comprehensive indexes to facilitate th looking up of various topics, they represent a library admirabl adapted to the requirements of either the technical or th practical reader.

¶ The Cyclopedia of Civil Engineering has for years occupie an enviable place in the field of technical literature as standard reference work and the publishers have spared n expense to make this latest edition even more comprehensiv and instructive.

¶ In conclusion, grateful acknowledgment is due to the staff of authors and collaborators—engineers of wide practical experience, and teachers of well recognized ability — without whose hearty co-operation this work would have been impossible.

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* For page numbers, see foot of pages.

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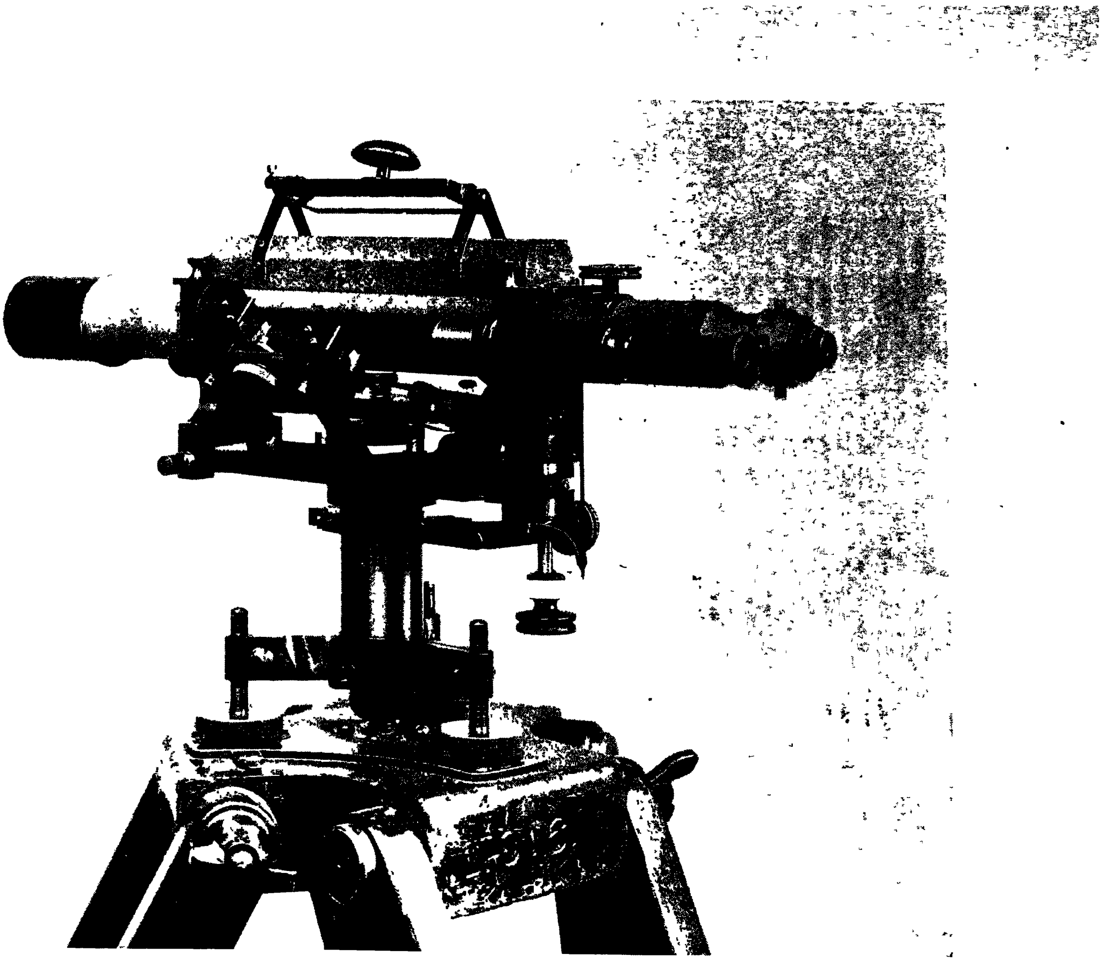


PLANE SURVEYING.

PART I.

Surveying is the art of determining, from measurements made upon the ground, the relative positions of points or lines upon the surface of the earth and of keeping records of such measurements in a clear and intelligent manner so that a picture (called plat) may be made of the lines or areas included in the survey. The records should be systematically arranged so that any person with a knowledge of surveying can use the notes intelligently. The field operations consist essentially of locating points, measuring lines and angles, measuring areas and laying out and dividing up areas. It is apparent that Arithmetic and Geometry are essential to the successful application of the principles of Surveying.

The subject may be divided into two parts: Plane Surveying and Geodetic Surveying.

In **Plane** Surveying, the portion of the earth included in the survey is regarded as a horizontal plane; in other words, the curvature of the earth's surface is neglected. In the ordinary operations of land surveying this assumption will not cause appreciable error as the lines and areas dealt with are of a limited extent.

As **Geodetic** Surveying, on the other hand, deals with extensive lines and vast areas, the effect of the curvature of the earth's surface must be taken into consideration.

All of the operations of surveying must proceed from the direct to the indirect. That is to say, we must first measure directly certain quantities upon the ground and from these calculate certain other quantities that cannot be measured directly. It is, therefore, apparent that all field measurements must be made with the utmost care, consistent with the nature of the problem involved, and that habitual inaccuracy and slovenly methods of keeping field notes must be avoided. Full details accurately measured and carefully and systematically recorded should be the

Measurement of lines. Probably the most elementary problem that presents itself is to measure the horizontal distance between two points without the use of instruments.

This can best be done by pacing, provided both points are accessible. In order to make this method of measurement efficient, it is necessary to determine as accurately as possible the length of one's pace. To do this, lay off upon firm, level ground by any convenient method, a line from 50 to 100 feet in length. Pass over this line from end to end, back and forth, keeping careful account of the number of steps taken each time the distance is covered. The total distance traversed, divided by the total number of steps will give the average length of one's pace. In thus ascertaining the length of the pace do not attempt to cover three feet at every step. It is better to adopt a natural, swinging gait.

Having thus determined the length of one's pace, the distance between two points may be measured approximately by walking in a straight line from point to point and counting the number of steps. This number multiplied by the length of step will give the length of line required. If the intervening space between the points cannot be traversed, as for instance when the two points are on opposite sides of a stream, the width of the stream may be ascertained approximately by stationing an observer on each side and noting the time elapsing between the flash of a pistol and the sound of the report. This interval, in seconds, multiplied by 1,090 (velocity of sound in feet per second) will give the distance in feet. Proper allowance must be made for direction and intensity of wind and therefore measurements of this kind had best be made upon a quiet day.

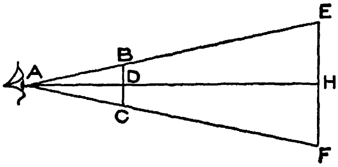


Fig. 1.

Another elementary problem frequently met with is as follows: Required to determine the altitude of an object such as a house or a tree, without the use of an instrument.

To solve this problem, take an ordinary lead pencil and hold it in a vertical position about two feet from the eye, the observer being far enough from the object for the visual angle intercepted by the pencil to just cover the object from top to bottom. The observer then paces the distance from his position to the object. The height of the object is determined as follows :

Let A, Fig. 1, represent the position of the observer's eye; BC the pencil held at the distance AD from the eye; EF the object whose height is to be ascertained. AH is the distance from the observer to the object and is to be paced. Then from similar triangles we have

$$BC : EF :: AD : AH, \text{ or } EF = \frac{BC \times AH}{AD}$$

For example, suppose the pencil is seven inches long and is held at a distance of two feet from the eye; the distance from the observer to the object being 85 feet. Then from the formula

$$EF = \frac{\frac{7}{12} \times 85}{2} = 24.8 \text{ feet nearly.}$$

In this, as in other problems, all quantities should be reduced to the same units.

The examples just given must be understood as illustrations merely and the student should avoid slipshod methods ; understanding that his best efforts will be needed in all surveying problems, and that the best is none too good.

SURVEYING WITH INSTRUMENTS.

Gunter's Chain, so called from the inventor, is well adapted to all classes of problems involving the calculation of areas from lines measured in the field. For many years this chain has been the English linear unit for all land measurements. It should be made of steel; it is 66 feet or 4 rods in length and has 100 links, each 7.92 inches. The handles are fitted with swivels to prevent the chain from kinking, and at every tenth link from either end is attached a brass tag with 1, 2, 3 or 4 prongs to assist in measuring. Thus the tag of four prongs indicates 40 links from one end, (See Fig. 2) but it represents 60 links from the other end; therefore care must be exercised in measuring, or distances may be measured from the wrong end of the chain. The 50-link mark is round in form so that it may be easily distinguished from the other tags.

and links; it is written thus: 15 chains and 82 links is 15.82 chains.

It is true that this chain is rapidly going out of use, yet one should be thoroughly acquainted with it, because many of the land records in this country are based upon it. In computing areas, the chain has the advantage that square chains are easily reduced to acres by simply moving the decimal point one place to the left; for example, a chain is 66 feet; the square would be 66 × 66 = 4356 square feet, which is $\frac{1}{10}$ of an acre. A rectangular lot having two sides of 6.32 and 2.15 chains respectively = 13.5880 square chains or 1.3588 acres.

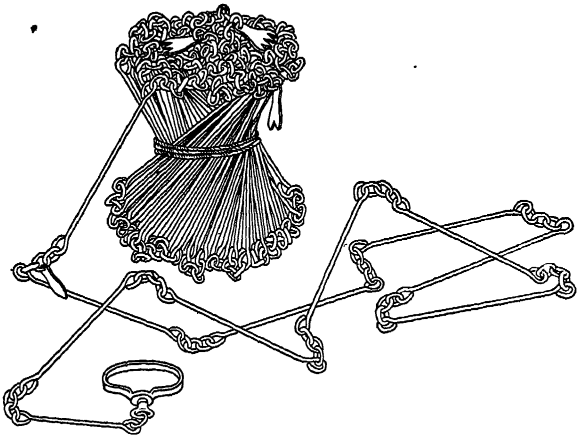


Fig. 2.

GUNTER'S OR LAND MEASURE.

7.92 inches	1 link
100 links or 66 feet or 4 rods	1 chain
10 square chains or 4 roods	1 acre = 43560 square feet
640 acres	1 square mile

A two-rod or half chain is sometimes used instead of the full Gunter's chain. Its only advantage is in the convenience of handling a shorter chain when working over uneven ground. Formerly the engineer's chain was almost universally employed in making surveys for surface canals, sewers, water-works systems, etc. It differs from the Gunter's chain in that is 100 feet in length and contains 100 links, each of which is, therefore, 1 foot long.

The unit of linear measure in the United States is the foot.

In measuring lines, a chain 100 feet long, divided into 100 links, is now in use. Distances are recorded in feet; decimals of a foot being used when possible. In cities where accurate and precise measurements are necessary, various kinds of tapes are used having the foot divided decimally.

It has been decided both by custom and law that the length of the boundary lines of a field is not the actual distance on the surface of the ground, but is the projection of that distance on a horizontal plane. The area of a field is not the exposed superficial surface, but as above stated, the projection of that surface on a horizontal plane. *For this reason, in all land surveying,* horizontal distances are to be measured and from these the areas computed.

The Gunter's chain, as well as the engineer's chain, is a very inaccurate device for measuring distances and areas unless special precautions are taken to counteract the errors to which it is liable. Some of these errors are cumulative and some compensating, and in what follows no attempt will be made to classify them. Some of the causes of errors will be pointed out and the surveyor should do all in his power to eliminate them.

The chain will sag between supports and thus the distance measured will be too short. This is sometimes allowed for by making the chain a given amount longer than the standard. Again, the chain may be standard under a certain pull and temperature, and for very precise work a spring balance is attached to one end of the chain to register the pull. A thermometer also is provided but is of little value from the fact that the temperature of the chain may vary considerably from that of the atmosphere. Still further, the length of the chain is likely to be increased from the wear of the links and connections. Each link with its connection has six wearing surfaces so that if each surface is worn away but $\frac{1}{100}$ inch the chain will elongate 6 inches. The rings and loops at the end of the links are frequently stretched out of the true form, thus elongating the chain; or the links may become bent, thus shortening the chain. In pulling the chain over the ground the links and rings have a tendency to collect weeds, mud, etc., and thus shorten the chain. In cold weather, ice and snow may collect in the joints with the same result. In using the chain, the links and rings kink and twist and a sudden jerk may break the chain. For these reasons the chain is not at present used as much as formerly.

The Tape. Tapes are made of various materials and are known as linen, metallic and steel.

Linen tapes, from the nature of the material, are likely to twist and tangle and when wet are easily stretched; for these reasons they do not long retain their standard length. They are used only in the roughest kind of work. Metallic tapes have a linen body with threads of copper or brass running throughout their length. These metallic threads prevent twisting and tangling and in a general way assist in preserving the standard length of the tape. They are better than linen tapes but not suitable for "good" work.

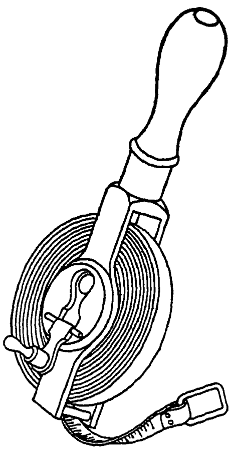
Steel tapes are of two kinds, "ribbon" and "band." Ribbon tapes are made of thin steel about $\frac{3}{8}$ inch wide. They are usually made in lengths of 50 or 100 feet. They are divided into feet, tenths and hundredths of a foot, the divisions being etched upon the tape. The other side of the tape is sometimes divided into rods and links to adapt it to land surveying, and it is either wound up into a leather case or upon a reel.

Ribbon tapes are generally used when considerable accuracy in measurements is required, such as laying out foundations for buildings, bridge piers, measuring up sewer lines, etc. From the nature of their construction, they will not stand much wear and tear, and are therefore not adapted to the rough usage of general field work. If carried in the case or reel, on account of the sharp bend at the center, the tape will soon break off at that point. After use in the field, the tape should be carefully wiped off and oiled if necessary, as the rust will obliterate the graduations and make it difficult to read. In using the ribbon tape in the field, care must be exercised to prevent twisting and kinking or catching under sticks or stones, as a slight jerk will break it.

The band tape is best adapted to general field work and to rough usage. It is made of heavy steel about $\frac{5}{16}$ of an inch wide and 100 feet long, divided into feet; usually the first and last foot are divided into tenths. The one-foot divisions may be marked by rivets, although the rivets tend to weaken the tape. They are sometimes marked by solder, which is notched at the proper point and stamped with the number. They are usually fitted with light, detachable handles for use in the field, but these are easily displaced or often lost in dragging the tape over stones or through grass. It is better to fit the tape with leather handles large enough to easily go over the hand. After use, the band tape should be gathered up in loops about three feet long and tied in the middle forming a figure eight. If it is desirable to wind the tape upon a reel, there are at present upon the market, several styles of reels, stiff in construction and convenient to carry.

The tape, like the chain, is likely to change in length due to changes of temperatures, and unless the proper pull is applied to the ends it will measure short of the standard. Altogether it is more accurate than the chain and of late years has largely replaced it for all kinds of field work. Indeed, with proper precautions, it has been found possible to obtain nearly as accurate results as with the most elaborate apparatus designed for measuring lines.

Since the methods of using the chain in the field are the same as for using the tape it will be sufficient to explain the methods of using the latter.



The Tape.

In connection with the tape there should be provided a set of eleven marking pins from 15 to 18 inches in length. To each pin should be attached a piece of red flannel to prevent its being overlooked in the grass. There should also be provided two rods (called flags), from 6 to 8 feet in length divided into foot lengths and painted alternately red and white. These rods are sometimes constructed of straight white pine, but $\frac{3}{4}$-inch gas pipe fitted with a steel shoe is better. It is desirable also, to provide a plumb-bob and string, and a hatchet.

Use of the Tape or Chain. For measuring a line with the tape, two men are required, a "leader" and "follower," or *head* and *rear* tapemen. The first step is to set one of the flags at the far end of the line to be measured, or if the line is too long, only as far ahead as can be distinctly seen. It is best to mark the beginning of a line with a stake driven as closely to the ground as circumstances will permit. The tape is then unrolled or unfolded in the direction of the line, the 100-foot mark going ahead. The leader takes the pins and the forward end of the tape and with a flag walks off in the direction of the forward end of the line, dragging the tape after him. When nearly one hundred feet away, the follower cries "down" and the leader faces the follower holding the flag vertically to be signalled into line by the follower. The tape is then stretched and straightened and a pin stuck vertically into the ground exactly at the 100-foot mark. The leader then picks up his end of the tape and starts off as before, the process being repeated each time, except that the follower must be particular to pick up each pin that is left in the ground by the leader.

If the line is more than eleven tapes in length, after the leader has stuck his last pin he cries "pins" and the follower delivers to him the ten pins that he has picked up. If the line to be measured is very long, some method should be adopted for keeping count of the number of times the pins have been exchanged. If the line ends with less than the length of a tape, the leader pulls out the tape to its full length, not sticking a pin, however, and then walks back and notes the distance from the last pin to the end of the line. This distance added to the number of pins held by the follower, including the last one stuck, will give the distance from the point at which the pins were exchanged. For instance, if the follower has six pins and the end of the line is 65 feet from the last pin, the entire distance from the point of exchange of pins is 665 feet. It must be remembered that each exchange of pins counts for 10 tape lengths or 1,000 feet.

Chaining on Slopes. One of the most important uses of the chain is to measure accurately distances where the surface of the land is uneven or of a sloping nature. In measuring up or down a slope, one end of the tape is raised until the top is as nearly as possible in a horizontal plane. If the slope is too steep to permit of one end of a full tape being raised enough to bring the tape horizontal, the tape is "broken," that is to say, only a part of the tape is used at each measurement. To do this the tape should be stretched to its full length, the leader returning to such a point upon the tape that the portion between himself and the follower may be properly leveled. A measurement is made with this portion, the operation being repeated with the next section of tape and so on until the entire tape has been used. Care should be taken not to confuse the pins. The high end of the tape may be transferred in any one of several ways, depending upon the degree of accuracy required. For great accuracy, a plumb-bob should be used but it should not be dropped and the pin placed in the hole made. It should be placed about where the bob will drop and the grass should be tramped down and the ground smoothed. The bob should then be lowered carefully until it almost touches the ground and allowed to come to rest. Then lower it until it reaches the ground when the pin should be stuck in the ground slantwise across the line exactly at the point of the bob. If less accuracy is permissible, it may be sufficient to drop the pin, ring end down and note where it strikes the ground, or a pebble may be dropped in the same way. In measuring uphill, the follower must hold the bob directly over the pin in the ground while he aligns the leader and sees that he sticks the pin while the bob is directly over the point in the ground. It is much easier to measure down than up hill so that when close measurements are required on slopes, the measurement should, if possible, be made down hill. Even under the most favorable conditions measuring lines with a tape is a most difficult operation for experts, and beginners cannot expect to attain efficiency except by constant practice and careful attention to every detail that will tend to eliminate error. For the method of chaining up and down hill, see Fig. 3.

Let it be required to find the distance A M, at which points two hubs have been established. Let A B C D E F, etc., be the points of the successive chaining and *a b c d*, etc., the horizontal planes. Starting from the point A; suppose the surface between A and B to be of no great difference in elevation, therefore, the full length of the chain can be used between these two points. The head chainman goes to the point B, and holds the head end of the chain on the ground, while the rear chainman holds the zero end at A and with aid of the plumb-bob "plumbs down," thus chaining the distance horizontal. This distance measured, the head chainman establishes a peg in the ground and calls out the distance "One hundred" or station one then goes to C, which must veniently plumb down to B. In this case the slope of the hill is greater than that between A and B, thus the impracticability of using the full length of the chain is apparent. This distance, therefore, is taken at a fractional part of the chain, as before stated, called "breaking the chain." The distances at such breaks should always be taken at an even number of feet whenever possible and at distances that are easily remembered, as 10, 20, 25, 50 feet, etc. The leader in every instance calls out the distance of such "breaks" and the rear chainman goes to the next peg and holds off the number of feet previously called out. Now as the distance A B is 100 feet and the distance B C 40 feet, the head chainman at C calls out "1 plus 40," meaning 140 feet from A. He next goes to D and the rear chainman calls out the distance measured "1 plus 40" and holds off 40 feet at *b*, and plumbs down to C. In this case the leader also plumbs down from *c* to D. This method is continued until M is reached, using the system of 1, 2, 3, 4, etc., plus the fractional measured distance, instead of using the whole number, as 125, 225, etc. The rear chainman should gather the pins after a new point has been established. As already stated the chaining can be checked by counting the pins picked up. Always allow the last pin to remain in the ground until absolutely certain it is no longer desired and can be of no further service. It is important that the distances should be checked by both chainmen as it may prevent serious mistakes, and in some cases prevent rechaining the entire distance. Distances in places where angles are taken are sometimes checked in the office by Trigonometry.

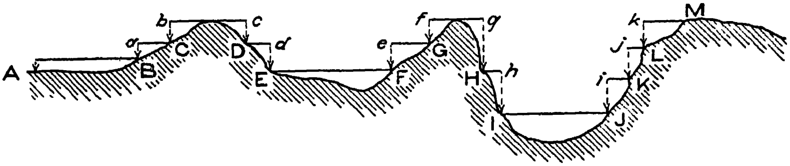


Fig. 3.

A few hints in regard to the use of the tape may not come amiss.

Always measure to and from the same side of a pin.

Hold the end of the tape as near the ground as possible.

Before sticking the pin, be sure there are no kinks in the tape and that the tape is not deflected to one side by grass, sticks or stones.

Never straighten the tape with a jerk; raise it clear of the ground and straighten and stretch with a steady pull and lower steadily into place.

The tape man should never brace himself against a pin; he should assume a position of stable equilibrium, preferably with one hand upon the ground.

In passing over uneven ground, every reasonable effort should be made to hold the tape level. Too much time should not be spent in attempting to hold the end of the tape exactly over the point in the ground when the difference of level of the ends of the tape is sufficient to neutralize what would otherwise be considered an accurate measurement.

In passing over rough ground the tape should be carried free from the ground, thus saving it unnecessary wear. The length of the tape is likely to vary from time to time, from changes in temperature, from constant stretching and from accident in the field For this reason the surveyor should compare frequently the lengths of his tapes with that of a standard. The length of the standard tape may sometimes be conveniently laid off upon the floor of a building, or two monuments may be set in the ground, the proper distance between them being measured either by a standardized tape or by means of wooden rods. Having found the error in the length of the tape the necessary corrections can then be made. If a line has been measured upon the ground, and it is afterwards found that the length of the tape is in error, the true length of the line may be found from the following proportion: the true length of the tape is to the length of the standard tape, as the true length of the line is to the length of the line as measured.

Suppose a line as measured, is found to be 625 feet in length and it is afterwards found that the tape is too long, by six inches.

Then we have: $100.5 : 100 :: x : 625$

from which x = true length of line = $628\frac{1}{8}$ feet.

EXAMPLES FOR PRACTICE.

1. A line as measured with a certain tape is 580 feet in

2. A line is known to be 840 feet in length, but when measured with a certain tape is found to be 842½ feet in length. Determine the true length of the tape.

Ans. 99.7 feet.

✓3. A certain field was measured with a Gunter's chain and found to contain 625 acres. It was afterwards found that the chain was ¼ foot too long. Determine the true area of the field.

Ans. 629.74 acres.

If an area has been measured with a certain tape that is afterwards found to be in error, the corrected area may be found by the following proportion: The square of the true length of the tape is to the square of the length of the standard tape as the true area is to the measured area.

Examples will now be given illustrating the use of the chain or tape, in the field.

1. *To erect a perpendicular at a given point in a line.* Let AB Fig. 4 be the given line and C the point in the line at which it is desired to erect a perpendicular. Since a triangle formed of the sides 3, 4 and 5 (or any multiple of these) will contain a right triangle, take parts of the chain or tape representing these distances or multiples and have the angle included between the shorter sides at C. Therefore, fasten one end of the tape or chain at E, 30 links or feet from C and the 90th link or foot at C. Then with the 50-foot mark in one hand, walk away from BC until both of the segments DE and DC are taut. Stick a pin or stake at D and DC will be the perpendicular required. If the perpendicular should be longer than can be laid out with the tape or chain, lay out CD as described and align a "flag" from C to D produced.

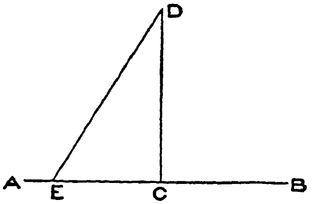


Fig. 4.

2. *To let fall a perpendicular to a given line from a given point outside the line.* (a) When the point is accessible:

Let AB Fig. 5 be a given line and C a point. From C as a center, with any convenient length of tape or chain as a radius, describe the arc DE, cutting the given line at points D and E. Stick pins at D and E and measure the distance. Bisect this distance at F; then CF will be the perpendicular required. If the line AB is too far from C to be reached with the chain or tape, it will be necessary to range out a line conveniently near to C which shall be parallel to AB. To do this erect at any convenient point on AB, as at N, Fig. 6, the perpendicular, and prolong it as far as necessary, as R. At R, erect RS perpendicular to RN. Then the perpendicular let fall from C upon RS and prolonged to AB will be perpendicular to AB.

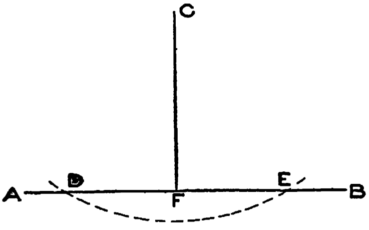


Fig. 5.

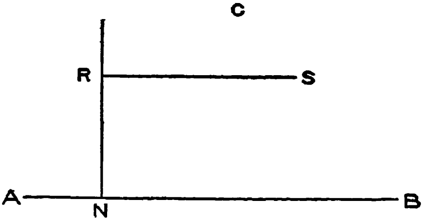


Fig. 6.

(b) When the point is inaccessible: Let AB, Fig. 7, be the given line and C the inaccessible point from which it is desired to drop the perpendicular to the line AB. At any convenient point F in AB erect the perpendicular FD and extend FD to E, so that FE = FD. Locate the point B so that B, D and C will be in the same straight line. Sight from E to C and find the point H in which this visual line crosses AB. Next find the point G at the intersections of DH and BE prolonged. Sight from G to C and the point M in which this visual line crosses AB will be the point required and the distance MG will equal MC. MC will be the perpendicular to AB at M.

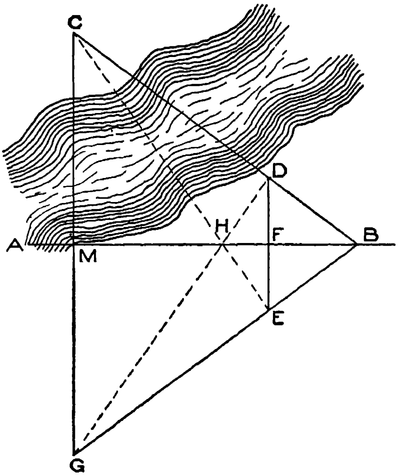


Fig. 7.

3. *Through a given point to run a line that shall be*

From point C let fall CD perpendicular to AB. At C erect CF perpendicular to CD; then EF will be the parallel required.

4. *To prolong a line beyond an obstacle.* Let AB, Fig. 9, be the given line which is intercepted by a tree, house or other obstacle. It is required to locate the line CD which will be in the direction of AB produced. At B erect BE perpendicular to AB of sufficient length to clear the obstacle and at E erect EF perpendicular to EB prolonging EF beyond the obstacle. At F and C erect perpendiculars to EF and CF making CF equal in length to BE, then CD will be the line required and the distance from A to D will equal AB plus EF plus CD.

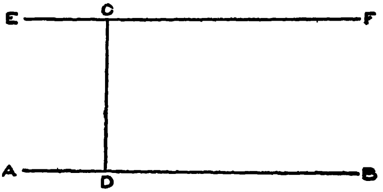


Fig. 8.

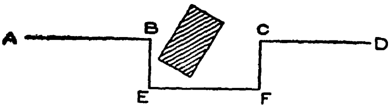


Fig. 9.

5. *When both ends of a line are accessible, but the line cannot be measured directly, on account of obstacles.*

At each end of the line erect perpendiculars of equal length sufficient to clear the obstacles, and measure the length of the line between the extremities of these perpendiculars.

6. *When both ends of a line are accessible, but neither can be seen from the other, thus preventing direct alignment.*

Such a case occurs when it is desired to run a line across a wooded field, the trees and underbrush preventing the alignment of the intermediate stations. Let AB Fig. 10 be the line whose length is desired. From A run a line AB′ (called a random line) in any convenient direction and continue it till the point B can be seen from B′. At B′ erect the perpendicular BB′ to AB′ and measure BB′. Then from the right-angled triangle ABB′ we will have

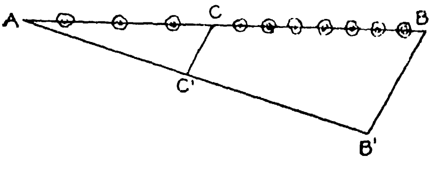


Fig. 10.

$$AB = \sqrt{\overline{AB'}^2 + \overline{BB'}^2}$$

The distance from A to any intermediate station as C can be found by measuring the length of the perpendicular CC′ to AB′ From similar triangles we have

$$AC : CC' :: AB : BB'$$

$$\text{or } AC = \frac{CC' \times AB}{BB'}$$

7. *To locate points in a line over a hill, both ends of which are visible from points near the summit.*

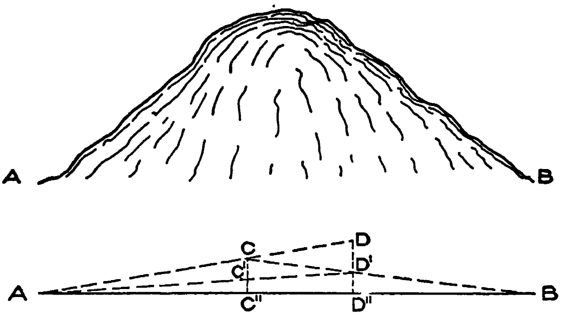


Fig. 11.

Set a flag at each of the points A and B Fig. 11. One man then goes to D, as closely in line with A and B as can be estimated. He then signals a man at C in line with A. C then signals D to D′ in line with B. D′ signals C to C′ in line with A and so on alternately until the points C″ and D″ are reached in line with A and B.

If the points A and B cannot be seen from the top of the hill, run a random line over the hill as described in problem 6 and offset to the true line.

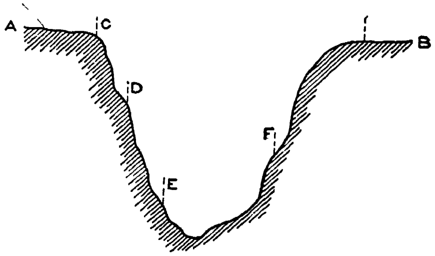


Fig. 12.

8. *To locate points in a line across a wide, deep valley, the extremities of the line being visible from each other.*

Fix a point C Fig. 12 on the edge of the slope in line with to B the intermediate points D, E, F and C can be put into line.

1. The Field Work of Measuring Areas. *Let us consider the triangular field ABC, Fig. 13.* Beginning at any convenient corner as A, measure from A to B, then from B to C, and finally from C to the point of beginning. Should a stream cut across the field as shown, measurements should be made from the corners to the points where stream crosses the boundary lines. Should it be found impossible to measure the sides of the field directly, owing to zigzag fences or other obstacles, offset parallel lines as in the figure and measure the length between such parallels.

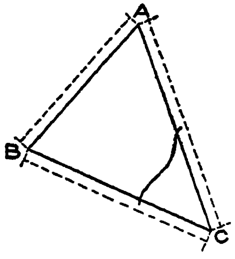


Fig. 13.

The area of the figure may be found from the following rule: From one-half the sum of the three sides, subtract each side separately. Multiply together the half sum and the three remainders and extract the square root of the product. This rule is explained in Art. 198 of Elementary Algebra and Mensuration.

If the lengths of the sides are given in chains, the area will be given in square chains. If the lengths of the sides are in feet, the result will be in square feet.

2. *To survey a four-sided field with the tape or chain.* Measure around the field in the same way as before, but in addition, it will be necessary to measure a tie-line between two opposite corners, thus dividing the figure into two triangles, the sum of whose areas will give the area of the entire figure. Such a tie-line is shown in Fig. 14 by the dotted line DB. If neither of the diagonals DB nor AC can be conveniently measured, measure the short tie-line LS for instance, and the distances CS and CL. Then in the triangle LCS the three sides are given, from which to find the angle LCS. Having this angle, we can calculate the length of DB and therefore the area of the two triangles composing the field.

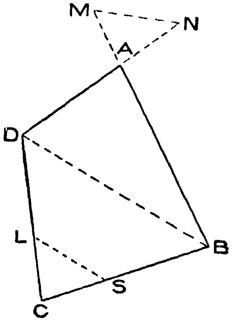


Fig. 14.

If it is not convenient to measure the tie-lines inside the field two adjacent sides as BA and DA can be prolonged to M and N forming the tie-line MN. It will usually be found more convenient to lay off CS equal to CL, thus forming an isosceles triangle.

3. *To survey a five-sided field with the tape or chain.* In this case two diagonals as EB and BD, Fig. 15, or two tie-lines as es and mn must be measured in addition to the lengths of the sides. Whatever the number of sides, a sufficient number of diagonals or tie-lines should be measured to divide the area into triangles from which the area of the entire field may be calculated.

If N represents the number of sides of a field, there will be required N–3 diagonals or tie-lines, forming N–2 triangles.

To simplify calculations when tie-lines are used in place of the long diagonals, the following method may be adopted:

Measure off Am any fractional portion of AE, and An the same fractional portion of AB and measure mn. Then mn will be to EB as Am is to AE or as An is to AB. Suppose for example that Am is $\frac{1}{10}$ of AE and An is $\frac{1}{10}$ of AB. Therefore EB is 10 times the length of mn.

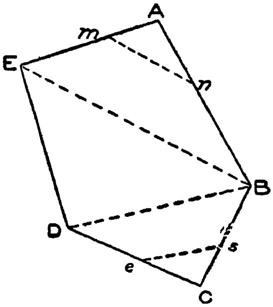


Fig. 15.

EXAMPLES FOR PRACTICE.

1. Given the three sides of a field as 5.25, 6.50 and 4.60 chains. Find the area of the field in acres, and square rods.

Ans. 1 acre, 31.46 square rods.

2. Given CB=3.65 chains, CD=2.85 chains, Cs=Ce=0.50 chains and es=0.65 chains.* Calculate the area of the triangle BCD.

Ans. 5.14 square chains area.

Off-sets and Tie-lines. To find the area of a field which is bounded in part by a stream, it is necessary to use off-sets, as follows: Measure the sides of the field in the usual manner and for the irregular boundary run a straight line, as ED, Fig. 16, and calculate the area of the field included between these boundary lines. To this area must be added the area included between the line ED and the irregular boundary.

To find this area, at points along ED, erect perpendiculars to the irregular shore line at such distances that the lines 1′ 2′, 2′ 3′ etc., may be considered straight. The desired area will evidently equal the sum of the areas of the trapezoids thus formed. The distance from E to any point 1, 2 or 3 on ED is called the *abscissa* of that point and the perpendicular distances from ED to 1′ 2′ 3′ etc., are called the *ordinates* of the point.

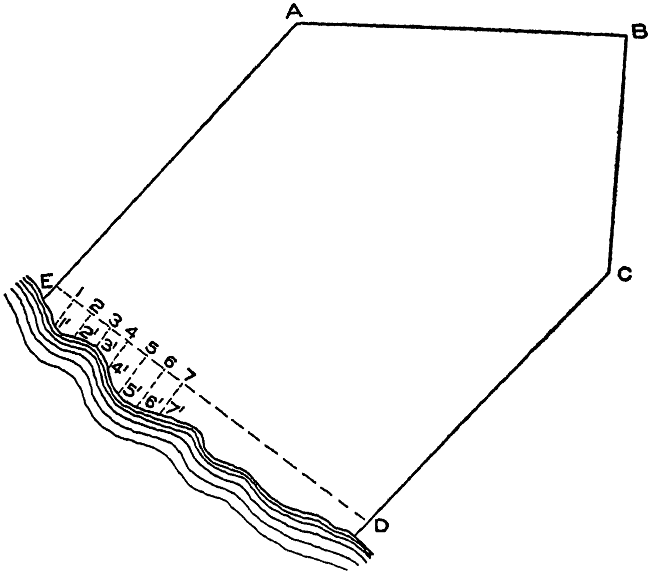


Fig. 16.

Instead of summing the trapezoid as above, the desired area may be found by the following rule: Multiply the difference between each ordinate and the second succeeding one by the abscissa of the intervening ordinate. Multiply also the sum of the last two ordinates by the last abscissa; one-half of the algebraic sum of these several products will be the area required.

To find the area of an inaccessible swamp, a lake or other area, run a series of straight lines entirely enclosing the given area, and since the diagonals cannot be measured, measure tie-lines either inside or outside of the area. As already stated, calculate the area included between the straight boundary lines and from this area substract the area included between off-sets let fall from points upon these boundary lines. Reference to Fig. 17 will make the method of procedure plain. Surround the inaccessible area by straight lines, AB, BC, CD, etc., and calculate the enclosed area. At proper intervals along these straight lines, erect and measure perpendiculars extending to the edge of the inaccessible area. Compute the area between these perpendiculars by the rule on page 20 and for the required area, subtract it from the area previously found. Since the long diagonals are not accessible, measure the area by measuring the interior tie-lines; remembering that the required number of tie-lines will be less by 3 than the number of sides enclosing the area.

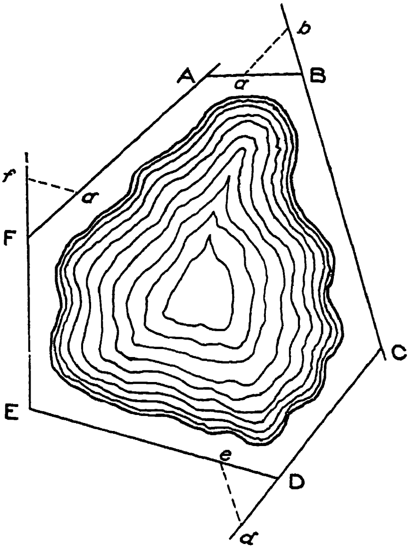


Fig. 17.

Example. Given the distances measured along the straight line AB (Fig. 18) with the corresponding off-sets measured to the broken line ACDE. It is required to compute the area between AB and the broken line ACDE.

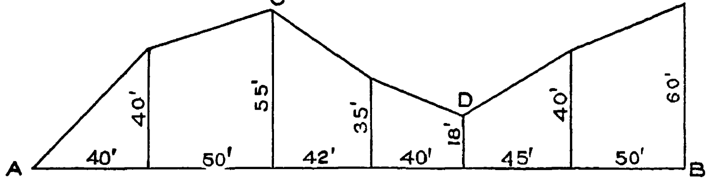


Fig. 18.

Difference of 1st and 3rd ordinates = 0′—55′=—55′ etc.
" " 2nd " 4th " =40′—35′=+ 5′
" " 3rd " 5th " =55′—18′=+37′
" " 4th " 6th " =35′—40′=— 5′

Abscissa of intermediate ordinates between 1st and 3rd= 40′×−55′=-2200
" " " " " 2nd " 4th= 90′×+ 5′= 450
" " " " " 3rd " 5th=132′×+37′= 4884
" " " " " 4th " 6th=172′×−5′=- 860
" " " " " 5th " 7th=217′×−42′=-9114
" " last ordinate = 267′×100′=26700

One-half the algebraic sum of the products as given above will give the required area.

$$\text{Area} = \frac{32034 - 12174}{2} = 9930 \text{ square feet.}$$

EXAMPLE FOR PRACTICE.

1. Given the distances measured along the straight line AB Fig. A with the corresponding off-sets measured to the broken line ACDEFB, to find the area between AB and the broken line ACDEFB. Check the result by calculating the areas of the trapezoids and triangles of the figure. Ans. 11,875 square feet.

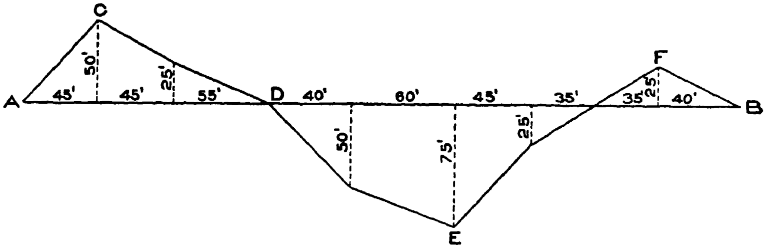


Fig. A.

Keeping the Field Notes. In keeping field notes, clearness and fullness should be constantly kept in mind. As field notes often pass into the hands of a second party, they should admit of but one interpretation to a person at all acquainted with the nature of the work. Extra time spent in the field in acquiring data will avoid confusion and vexatious delays when the notes are worked up in the office. Avoid the habit of keeping notes upon scraps of paper or in vest-pocket note books. Provide note books especially adapted to the keeping of field records and number and index them so that the contents may be understood at a glance. Remember that sketches made upon the ground aid materially in interpreting field notes that otherwise might be unintelligible.

There are three principal methods of keeping field notes; first, supplemented by sketches. The third method is without doubt the best, but examples of the others will be given. For keeping the notes of the chain survey there should be provided what is known as a field book, a pencil (preferably 4H), rubber eraser and and a short rule for drawing straight lines.

First, by Sketches Alone. Either page of the note book may be used for sketching but it will be more convenient to use the right-hand page, as it is ruled into squares, thus permitting sketching to scale. Always sketch in the direction of the survey, beginning at the bottom of the page and making the center line of the page correspond approximately with the North and South lines.

Second, by Notes Alone. Use the left-hand page of the note book beginning at the bottom as before. Do not crowd the notes, and if necessary use two or more pages. See Fig. 19.

Fig. **19** shows the method of keeping the notes of the survey shown in Fig. 20.

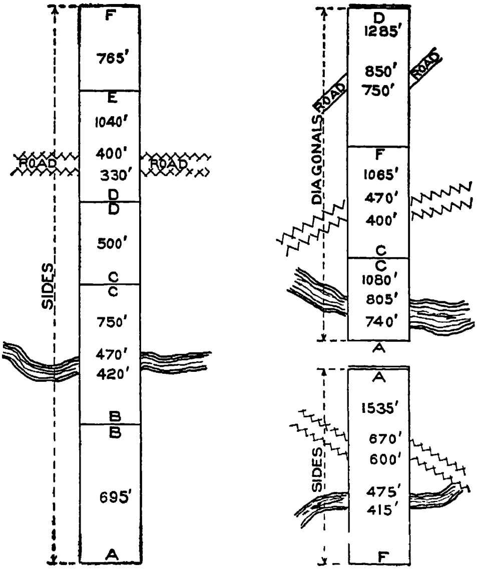


Fig. 19.

Third, by Notes and Sketches. It is apparent that in this method both the first and second methods are embodied in the notes.

THE VERNIER.

The vernier is an auxiliary scale for measuring with greater precision the spaces into which the principal scale is divided. The smallest reading of the vernier, or the least count, is the difference in length between one division on the main scale and one on the

Abscissa of intermediate ordinates between 1st and 3rd= 40′×—55′=-2200
" " " " " 2nd " 4th= 90′×+ 5′= 450
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" " " " " 5th " 7th=217′×—42′=-9114
" " last ordinate = 267′×100′=26700

One-half the algebraic sum of the products as given above will give the required area.

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EXAMPLE FOR PRACTICE.

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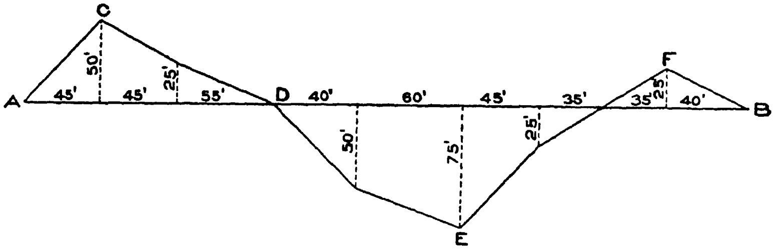

Fig. A.

Keeping the Field Notes. In keeping field notes, clearness and fullness should be constantly kept in mind. As field notes often pass into the hands of a second party, they should admit of but one interpretation to a person at all acquainted with the nature of the work. Extra time spent in the field in acquiring data will avoid confusion and vexatious delays when the notes are worked up in the office. Avoid the habit of keeping notes upon scraps of paper or in vest-pocket note books. Provide note books especially adapted to the keeping of field records and number and index them so that the contents may be understood at a glance. Remember that sketches made upon the ground aid materially in interpreting field notes that otherwise might be unintelligible.

There are three principal methods of keeping field notes; first, by sketches alone; second, by notes alone; and third by full notes supplemented by sketches. The third method is without doubt the best, but examples of the others will be given. For keeping the notes of the chain survey there should be provided what is known as a field book, a pencil (preferably 4H), rubber eraser and and a short rule for drawing straight lines.

First, by Sketches Alone. Either page of the note book may be used for sketching but it will be more convenient to use the right-hand page, as it is ruled into squares, thus permitting sketching to scale. Always sketch in the direction of the survey, beginning at the bottom of the page and making the center line of the page correspond approximately with the North and South lines.

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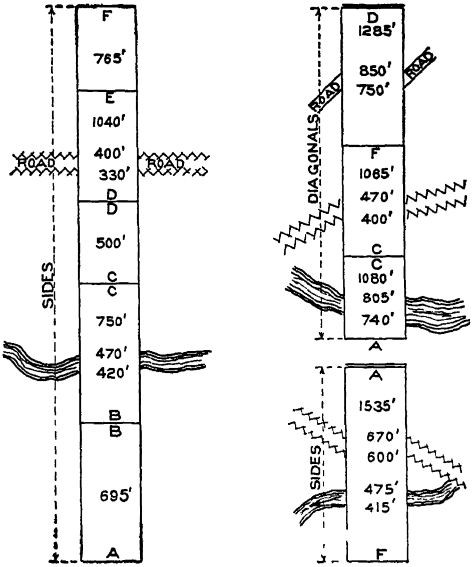


Fig. 19.

Third, by Notes and Sketches. It is apparent that in this method both the first and second methods are embodied in the notes.

THE VERNIER.

The vernier is an auxiliary scale for measuring with greater precision the spaces into which the principal scale is divided. The smallest reading of the vernier, or the least count, is the difference in length between one division on the main scale and one on the vernier.

A vernier is said to be **direct** when the divisions on the vernier are smaller than those on the main scale Fig. 21 A; **retrograde,** when the divisons on the vernier are greater than those on the main scale. See Fig. 21B.

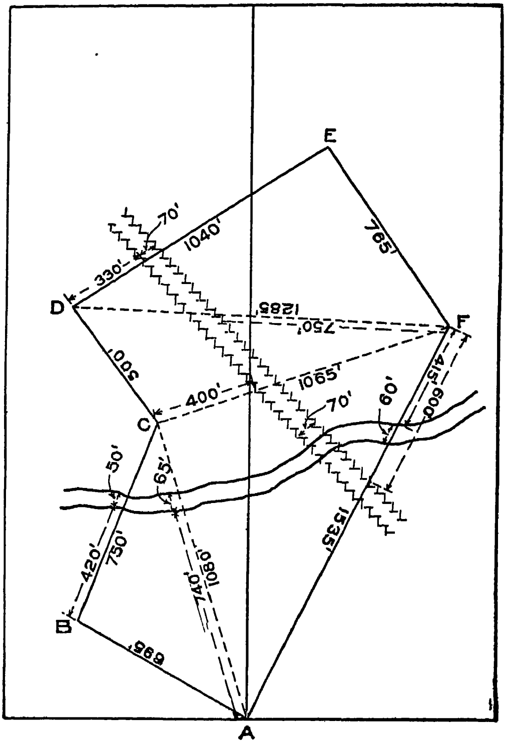


Fig. 20.

In Fig. 22 let MM represent a scale divided into tenths;. then since ten spaces on the vernier VV are equal to nine spaces upon the scale, it is evident that each space upon VV is short by one-tenth of a space of MM. The least count is therefore, $\frac{1}{10}$ of $\frac{1}{10}$ or $\frac{1}{100}$.

The vernier and slow motion screw of the vertical arc of the engineer's transit are attached to the left hand standard of the instrument.

Fig. 23 represents a vernier as applied to an engineer's transit. It will be noticed that the main scale is divided so as to read directly to 30 minutes. The vernier is so divided that 29 spaces upon the main scale equal 30 spaces upon the vernier, therefore the least count of the vernier is $\frac{1}{30}$ of 30 minutes or 1 minute.

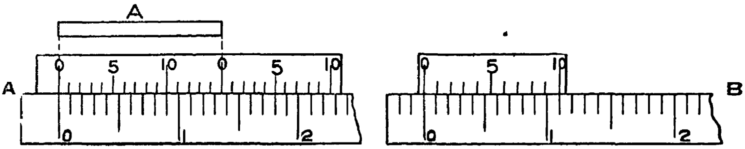


Fig. 21.

It will be apparent, therefore, that the readings are taken in the direction of the increasing graduations of the main scale has passed the 156th space on the main scale, and is near the 30 minute (half degree) division *b*, therefore the coinciding lines of the vernier and main scale must be between 0 and 30′, and we

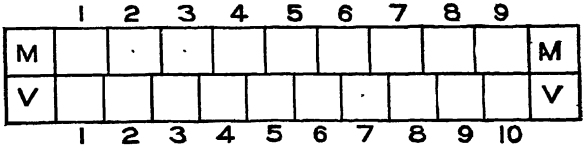


Fig. 22.

find them, by looking along the scale of the vernier, at 17 minutes hence, the reading is 156° 00′+17′=156° 17′.

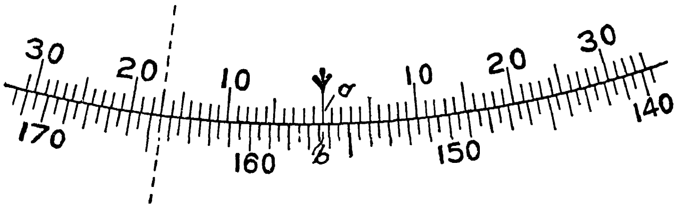


Fig. 23.

Fig. 24 represents another method of division of the circle of the transit. The vernier is double, and the figures on the vernier are inclined in the same direction as the figures on the scale to which they belong.

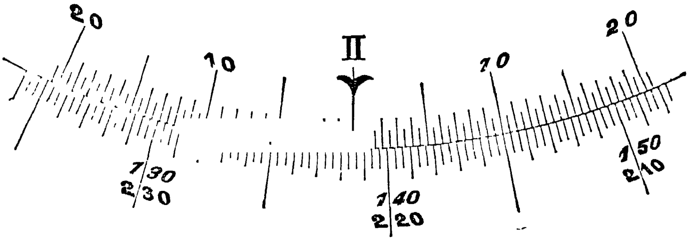


Fig. 24.

It will be noticed that the main scale reads directly to 20 minutes and that the vernier is so divided that 39 spaces upon the scale correspond to 40 spaces upon the vernier. The least count of the vernier is therefore $\frac{1}{40}$ of 20 minutes or $\frac{20}{40}$ of 1 minute equals 30 seconds.

vernier is beyond the 138° mark and about half way between th first and second 20′ divisions. The reading so far is then 138° 20′ Now look along the vernier to the right until a line upon the vernier is found that seems to be a prolongation of a line upon th scale. This occurs at the division marked 10 upon the vernie so that the reading is 138° 20′ + 10′ or 138° 30′.

For the outside scale, the zero of the vernier is beyon the 221° mark and about half way between the first and secon 20′ divisions. The reading so far is therefore 221° 20′. Nov look along the vernier to the left as before, and the divisions coincide at the division marked 10 upon the vernier, so that the reading of the outside scale is 221° 20′ + 10′ or 221° 30′. The sun of the readings of the two scales equals 360° as it should.

EXAMPLES FOR PRACTICE.

1. Determine the least count of the vernier in Fig. A, 3 spaces upon the scale, being equal to 40 spaces upon the vernier

2. Determine the least count of the vernier in Fig. B, 5 spaces upon the scale being equal to 60 spaces upon the vernier The figure represents what is called a *folding vernier*. To read i follow along the vernier in the usual way until the division marke 10 is reached. If there are no corresponding lines, then go bac to the other end of the vernier beginning with the other 10 mar and follow it back toward the center of the vernier.

3. Determine the least count of the vernier of Fig. C, whic represents the usual method of dividing the vertical circle of th transit.

The Level Bubble is one of the most important attachments o an engineering instrument, and an instrument otherwise good ma be rendered useless by imperfect level tubes.

The spirit level is a glass tube nearly filled with a mixture o ether and alcohol, the remaining space being occupied with th vapor of ether. Alcohol alone has not proved satisfactory as it i too sluggish in its movements, thereby rendering an instrumen lacking in sensitiveness. If the tube were perfectly cylindrical the bubble would occupy the entire length of the tube, when horizontal, or when slightly inclined to the horizon, thus rendering i impossible to tell when the tube is in a truly horizontal position

The tube is, therefore, ground on the inside so that a longitudinal section is a segment of a circle. If the tube is not ground to an an even curvature the bubble will not travel the same distance for every minute of arc to the extreme ends of the tube, and an otherwise perfect instrument will not work well.

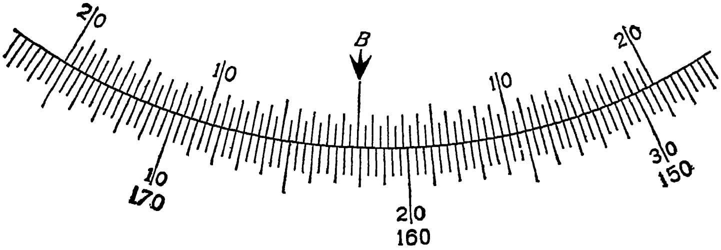


Fig. A.

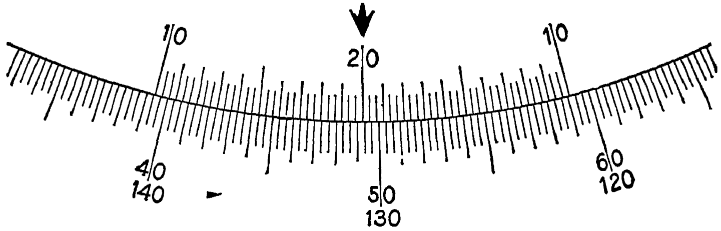


Fig. B.

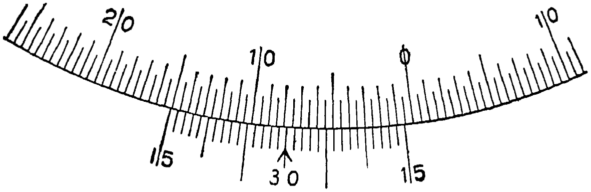


Fig C.

A line tangent to the circular arc at its highest point, as indicated by the middle of the bubble, or a line parallel to this tangent, is called the axis of the bubble tube. This axis will be horizontal when the bubble is in the center of the tube. Should the axis be slightly inclined to the horizontal, the bubble will move toward the higher end of the tube, and the movement of the bubble should be proportional to the angle made by the axis with the horizontal. Therefore if the tube is graduated, being a portion of the circumference of a circle with a radius so long that the arc of a few sec onds is of an appreciable length, it will be possible to determine the angle that the axis may make at any time with the horizontal, provided the angular value of one of the divisions of the tube is known. This is done by noting by how many divisions the center of the bubble has moved from the center of the tube.

Since divisions of uniform length will cover arcs of less angular value as the radius of the tube increases, and since a bubble with a given bubble space will become more elongated as the radius is increased, the sensitiveness of the bubble is proportional to the radius of curvature of the tube and the length of the bubble. The length of the bubble, however, will change with changes in temperature, becoming longer in cold weather and shorter in warm weather. This is due to the fact that the liquid in the tube expands and contracts more rapidly than the glass. If the bubble contracts excessively, the sensitiveness is thereby impaired, and it should be possible to regulate the amount of liquid in the tube. This is done by means of a partition at one end, having a small hole in it at the bottom. A bubble should come to rest quickly, but should respond easily and quickly to the slightest change of inclination of the tube.

To determine the radius of curvature of the tube, proceed as follows: Let S = length of the arc over which the bubble moves for an inclination of 1 second. Let R = its radius of curvature.

$$\text{Then } S : 2\pi R :: 1'' : 360^\circ.$$

$$\text{From which } R = 206265 \times S \quad \text{Or} = S = \frac{R}{206265}$$

S may be found by trial, the level being attached to a finely divided circle. Or, bring the bubble to the center and sight to a divided rod; raise or lower one end of the level and again sight upon the rod. Call the difference of the readings h, the distance of the rod d, and the space which the bubble moved S. Then from approximately similar triangles

$$r = \frac{d\,S}{h}$$

EXAMPLE FOR PRACTICE.

1. At 100 feet distant, the difference of readings was 0.02 foot, and the bubble moved 0.01 foot. What is the radius of the bubble tube?

Locke's Hand Level. This instrument consi six inches long with a small level mounted on it of the center near the object end. See Fig. 25. level is an aperture across which is stretched attached to a frame. This frame is made adj and a spring working against each other, or by screws placed at the ends of the level mounting. In th directly below the level, and at 45° to the line of sight, is placed a totally reflecting prism acting as a mirror. The images of the bubble and wire are thus reflected to the eye. The prism divides

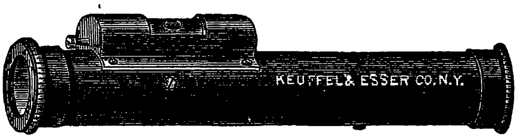

KEUFFEL & ESSER CO. N.Y.

Fig. 25.

the section of the tube into two halves, in one of which is seen the bubble and wire focussed sharply by a convex lens placed in the draw tube at the eye end of the instrument, while the other permits of an open view. Putting the instrument to the eye and raising and lowering the object end until the bubble is bisected by the horizontal wire, natural objects in the field of view can be seen through the open half at the same time, and approximate levels can then be taken. To prevent dust and dampness from entering the main tube, both the object and the eye ends are closed with plain glass.

There are two adjustments necessary in this instrument: First, the bubble tube: it should be so adjusted that the bubble will be in the center of the tube when the instrument is horizontal. Second, the horizontal wire; it should bisect the bubble when the latter is in the center of the tube. The methods of executing these adjustments are so apparent it will be unnecessary to dwell upon them here.

The instrument is intended to be carried in the pocket and is of especial value upon reconnaissance surveys, and for sketching in topography upon preliminary surveys.

For topographical purposes, the topographer should provide painted alternately red and white. Upon this rod, the topographer should mark by a notch or other means, the height of his eye above the ground. Standing then upon a station of the line of survey, the topographer directs his assistant to carry the rod out upon either side of the line and in a direction at right angles thereto, until a point having the proper elevation above or below the center line is found as determined by the topographer holding the instrument in a horizontal position at the eye. The topographer then paces the distance, while the assistant carries the rod to the next point. It is evident that if the line of sight from the instrument coincides with the mark upon the rod, the two points upon the ground are at the same level. If the line of sight strikes the rod, say one foot below the mark upon the rod, it is evident that the ground where the rod is held is one foot higher than where the instrument is held. These operations can be repeated indefinitely and made to extend as far as necessary upon either side of the line. The points of proper and equal elevation are then connected forming contour lines, but the topographer should fill in details by the eye. The methods of keeping the field notes will be illustrated and described later.

Let BCXDEFG and H, Fig. 26, represent the successive rod readings on the right of the center line A, and B′ C′ D′ E′ F′ G′ the readings on the left. Now suppose the leveler stands with a Locke-level at zero and the rod is held vertically at B. The line of sight *ab* bisects the rod at 8.6 feet. The distance from the ground to the observer's eye is 5.5 feet. Thus it is apparent the elevation at B will be 3.1 feet lower than at A. The observer now paces the distance between A and B, and finds it to be 50 feet. The reading is now taken at C on the line of sight *cd* which reads 6.2 feet, hence the elevation of C is .7 foot lower than B, and the distance between 20 feet. Suppose an attempt is made to take a reading near D. Since the horizontal plane from the observer's eye to the ground does not strike the rod, it is apparent that the rod is too far away, therefore it should be moved back to a point X where the horizontal plane *ef* will bisect the rod at some division. The elevation of X having been ascertained, pace the distance CX, as in the former cases. This method is continued until H is reached, the distance between each. The same method is used on the left-hand side of the center line. However, where the surface of the ground has an abrupt change between stations, it is customary to take cross sections at such changes and ascertain the distances between the stations by pacing; the center line at such points is accepted as zero; in the same manner perform the operation as if at a station. Where a cross road intersects the center line or any portion of the cross section, take readings at places that show an abrupt change, as the top of a bank, side of the road, or gutter, center of the road and on the other side in the same way and place as before. This rule holds good in places where small streams are situated. It is not necessary to find the depth of the water, because the purpose of the cross section deals solely with the surface. Where obstacles prevent the section being run at right angles to the center line, use the method of off-sets and secure the desired elevation as closely approximate as circumstances will permit.

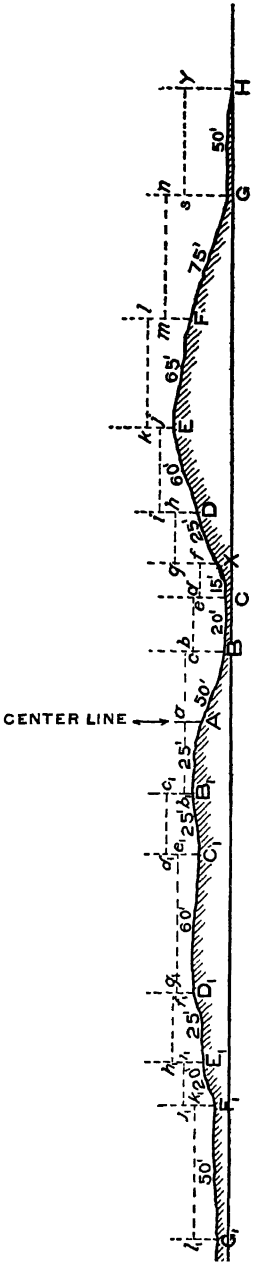


Fig. 26.

The Abney Hand-Level and Clinometer. This instrument is similar to the Locke hand-level, see Fig. 27, but the small spirit level mounted on top can be moved in the vertical plane and is clamped to a dial graduated upon one side into single degrees and upon the other into slope ratios, so that it is possible to measure angles of slope.

The adjustments of the instrument are the same as for the Locke hand-level.

in the same manner as the Locke hand-level, but is of more universal application. It is of especial value upon steep slopes when the efficiency of the Locke level would be limited by the length of rod. In using the Abney instrument it is only necessary to mark the height of the eyes upon the rod. In sighting upon the rod, with the horizontal line coinciding with the mark upon the rod, move the vertical circle until the bubble is in the center of the tube. Read the vertical angle, and the tangent of this angle multiplied by the horizontal distance to the rod will give the difference of elevation. If the distance to the rod is measured along the slope of the ground, multiply this distance by the sine of the vertical angle to get the difference of elevation.

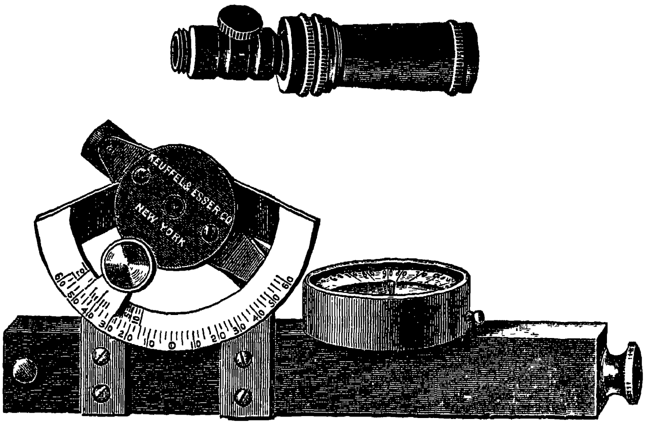


Fig. 27.

The most satisfactory method of using this instrument is in connection with a straight edge from 8 to 10 feet in length. The straight edge is laid upon the ground parallel to the direction of slope and the clinometer is then applied to it, the vertical circle being turned till the bubble is in the center. The angle of slope is then read, or better still, the slope ratio is read from the vertical circle. This operation is repeated at every change of slope, the distances being either paced or measured with a tape. For instance, suppose the slope is found to be 60 feet in length and the slope ratio as given by the clinometer is $\frac{1}{10}$. It is evident then that at the end of the slope the difference in elevation will be 6

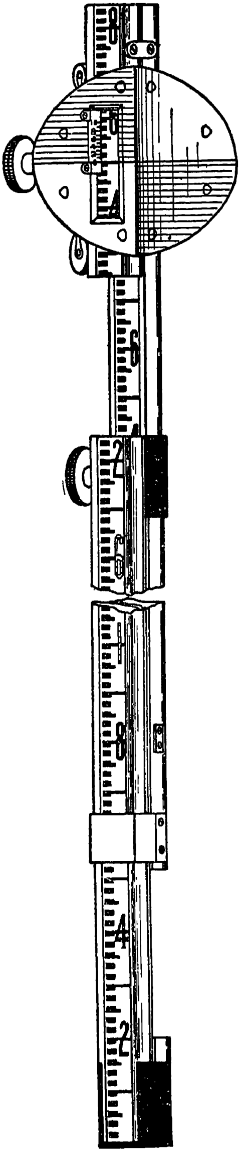


Fig. 28.

feet. The instrument is sometimes fitted with a small compass and a socket for use upon a tripod or Jacob staff.

The Leveling Rod is an important part of the leveling outfit; it is used in measuring the vertical distance between the horizontal plane through the line of sight and the point upon which the rod is held. There are three forms in common use known as the New York, Philadelphia and Boston. They are made of hard wood 6½ feet long, sliding out to 12 feet and provided with target, vernier and clamps.

Leveling rods are of two kinds, the target and the self-reading. Of the target rods, the New York and Boston are generally used for precise work. Of the self-reading rods, the Philadelphia shown in Fig. 28 is in more common use. The self-reading rods are used only in connection with that class of work where approximate accuracy only is required; this form is generally read to hundredths of a foot and can be read directly from the instrument by the observer without the aid of the target, as is suggested by the name. However, with the aid of the target this rod can be read to thousandths of a foot approximately. The target is used when greater accuracy is required and when the rod is so far from the instrument that it cannot be distinctly read.

The rod consists of a graduated scale divided into feet, tenths and hundredths of a foot, and when properly made, readings to thousandths of a foot can be easily taken. The numbers making the tenths should be 0.06 foot long and so placed that one-half the length is above and one-half below the line. The numbers marking

This class of rod is painted white, the foot graduations are red and the tenths and hundredths are black horizontal lines.

No attempt will be made to describe the reading of the vernier of either the New York or Boston rod, but the Philadelphia rod is so divided as to make its reading easily understood. With this rod each side of the black horizontal line indicates 100ths, that is, the lower side of the first black space is called "one," and the upper side of the same space is called "two," the lower side of the third space is called "three" and so on until the tenth is read.

The reading is taken without the aid of the target, in feet, tenths and hundredths as the case may be. The movable target has a vernier which reads to thousandths of a foot and is read from zero to ten. To read this rod, move the target to any convenient place on the scale of the rod and note where the vernier at zero coincides with a black horizontal line; then note where a line of the vernier coincides with a line of the scale. For example, if the zero of the vernier is just above one foot, four-tenths and five hundredths, as shown in Fig. 29, and a line of the graduation of the vernier coincides at 7 with a horizontal black line on the rod, the reading will be 1.457 as is shown in Fig. 29. If reading to the nearest 100th, the reading will be 1.46. This is because the 7 naturally brings the zero .002 above the line of graduation on the rod, therefore, the zero of the vernier is .002 nearer the 6 than the 5, hence, the reading is as above. Should the vernier read .002 instead of .007 the reading would be 1.45. It is apparent that .002 now brings the zero of the vernier below the line hence it will

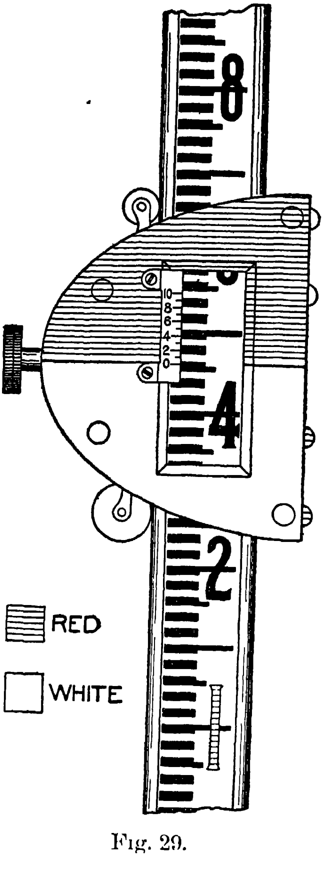


Fig. 29.

MAPPING THE MESCALERA INDIAN RESERVATION, NEW MEXICO
Triangulation signal and party on Sierra Blanca, 12,000 feet above sea level.
Courtesy of United States Geological Survey, Washington, D. C.

rod reading is 1.45. Therefore, in all readings with the Philadelphia rod, read the thousandths to the nearest half hundredth. This is true whether or not the lines coincide.

These readings apply only to the face of the rod or to 6½ feet. When the rod is extended to 12 feet, or any fractional part thereof, the reading is a little different, both as to its graduation and vernier. The scale, of course, is the same on the face of the rod when extended, except as to the vernier, which is placed on the back at 6½ feet and the scale of graduation on the extended part of the rod is also on the back of the extension which runs through the vernier, as shown in Fig. 30. The scale of hundredths is the only part to be particularly observed, together with the vernier in the former.

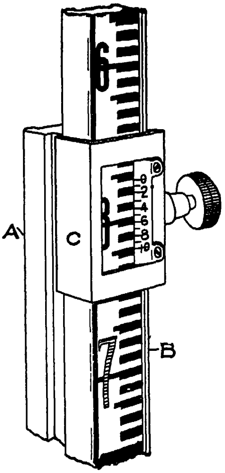


Fig. 30

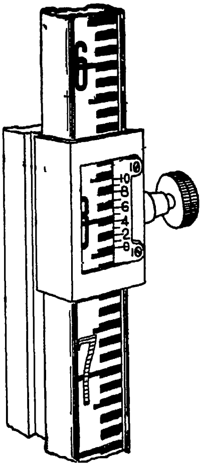


Fig. 31.

For example, the first horizontal black space equals "one," which is the top line of the foot mark. The lower side of the first black space is "two," and the upper side of the same space is "three" hundredths, and so on until the tenth is reached. The tenth and feet are placed the same as on the face of the rod. The vernier, as already stated, is a little different in point of reading and is graduated from ten to zero, instead of zero to ten, as on the movable target. However, with some recently-made rods of this type, the scale and vernier reading is the same throughout. See Fig. 31. The graduation at ten is taken as the zero in determining thousandths. The vernier in question is firmly attached to the upper end of the rod 6½ feet, (and the extension of the rod runs through this vernier). The differences in graduation of the two sides should be carefully noted. The rod has two clamp screws, one attached to the movable target and the other near the vernier on the back of the rod. In running the rod, it desired, to set the target at 6½ feet and run the rod to its full length, then move down as signalled; where no target reading is required, run the rod to its full extent (12 feet) and as the face of the rod has a scale throughout, the reading can be taken from the instrument.

Should the instrument not be near enough to enable the leveler to see the rod distinctly without the aid of the target, he should first read the rod through the telescope of the instrument and then notify the rodman at what distance the intersection of the cross-hairs in the instrument approximately bisects the rod, such as 3.21, which means three feet, two tenths and one hundredth. The rodman then sets his rod to read this distance and another sight is taken, being careful to have the rod plumb. Should the intersection of the target fail to coincide with the cross-hairs in the instrument, the leveler then signals, or calls out if sufficiently near to do so, the true rod reading, as up a tenth, down two hundredths, as the case may be, and the target is placed at this distance. When precision is required, this method is relied upon only for the approximate placing of the target; the method used in this case is to slowly move the target by standing behind the rod and holding it between the thumb and fingers of one hand, while the target is moved with the other. Then the target is slowly moved by the signals of the observer.

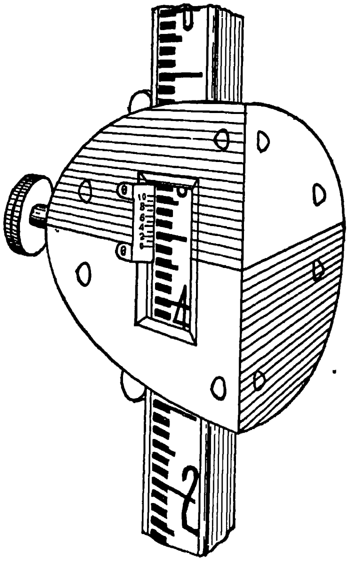


Fig. 32.

When a slow motion with the hand above the shoulder or below the hip is made by the observer, it means that the rod is to be moved in that direction a fractional part, as one tenth. but when a quick motion is made and the hand drawn back in the same manner it implies that the target is to be moved just a trifle. In this way and by proper attention to the signals of the thereby saving much time. When the target is finally set, the rodman reads the rod and calls out the reading to the observer, when within reasonable distance. The target rods are read entirely by the rodman, and the readings are kept by him in a note book for that purpose; these notes should be given to the observer

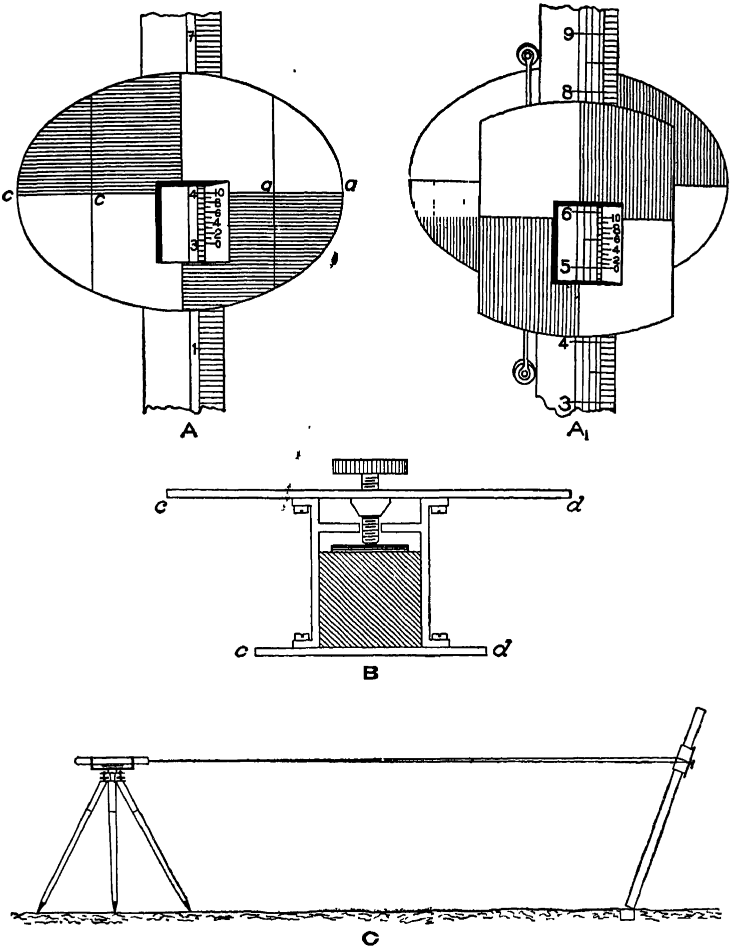


Fig. 33.

at every opportunity and results checked. To obtain correct results when leveling, it is absolutely essential that the rod be vertical and the rodman should remember to hold the rod in this

The observer or leveler, by means of the vertical wire upon the target of an ordinary leveling rod, can tell whether or not a rod is vertical and in a position at right angles to the line of sight, but he is not able to determine whether the top of the rod is inclined towards the instrument or in the opposite direction; because when looking through the telescope of a level he can see only a fractional part of the rod. Therefore the necessity of overcoming this difficulty led to the invention of the bent target which obviates this latter trouble as can readily be seen from Fig. 32. The American target fulfills the same requirements, but differs from the ordinary target in having two discs, one behind the other, as in Fig. 33. The principle of construction of this target is extremely simple, and may be best explained in the figures above Suppose a target of the old kind, which in its front view looks exactly like the front view of the new target in A, to be cut along the vertical lines *aa*, *bb*, thus dividing it into three parts; that is, one center-piece and two wings. Suppose furthermore, the centerpiece to remain in its former place at the front of the rod, while the two wings are removed to the rear of the rod. Then the result evidently will be that the horizontal line *cc*, *dd*, will appear as one unbroken line to the observer, only when the rod is held perfectly vertical. Any deviation either towards the instrument, or away from it will cause the two parts *cc* and *dd* of the horizontal line situated at the rear of the rod in the wings of the target to show either above or below that part *cd* of the horizontal line which is situated in the front of the rod in the centerpiece of the target.

Whether using the bent target, the ordinary target or no target at all, it is apparent that the rodman should hold the rod in a vertical position, known as plumb. This can be done by standing directly behind the rod with both feet together or apart, as the rays of the sun may require, governing shadows, and holding the rod between the thumb and finger of one hand while moving the target with the other. After the target is set, both hands are brought in line with the shoulders, and standing erect, the hand should touch the rod very lightly, so that it will almost stand in a vertical position by itself; or when standing in that position if the

with the nose and chin, it will be plumb. The rod
never put his hands around the rod.

Another way is to sight along the line of som
apparently in line. There are, of course, many different
of signalling, but the ones mentioned are frequently used.

When the rod is to be read without the aid of a target or with the ordinary target, it frequently happens that the rod is not vertical and the signal used for bringing it in its proper vertical plane is the raising of either the right or left hand in a vertical position, which indicates that the rod should be inclined in that direction. In so doing move the rod slowly until the hand is lowered. After the target has been set in its proper position, clamp it by the screw on its side, then give another sight and note the signals of the observer.

If the target is to be moved, the observer should hold the palm of one hand in the direction the target is to be moved. The observer should use but one hand in signalling the rod; if the target is to be lowered, he should hold his hand below his hip, palm down. To raise the target he should hold his hands above his shoulder, palm up. Any considerable change in the position of the target is denoted by a more or less violent motion of the hand. If a very slight change is desired, the observer should hold his hand in the proper position without moving it up or down. When the proper position of the target has been obtained, the observer indicates the fact by raising both arms above the head and moving them in the arc of a circle to indicate that the rod reading at that particular place is complete.

New York Rod. This rod resembles the Philadelphia rod as to its use and dimensions, but differs as to scale and vernier reading. The scale is divided into feet, tenths and hundredths, the same as the Philadelphia rod, except the graduations of the hundredths, which instead of having the sides of one black space each equivalent to 0.01 foot, as on the Philadelphia rod, the hundredths are distinct by themselves, therefore, each line between the tenth is .01 part of the scale. Fig. 34 shows the rod at its full length, and Fig. 35 shows a sectional part thereof with its movable target set at 6¼ feet and the black horizontal lines each indicate .01 as

The rod has a movable target which carries a vernier, and allows readings to thousandths. It cannot, however, be read without the aid of the target and is used for the most part, where precision is required; it probably commends itself to a greater number of engineers, because of its stiffness and wearing qualities. For elevations up to $6\frac{1}{2}$ feet, the target is used in the same manner as the former rod by sliding it up or down upon the rod. Above $6\frac{1}{2}$ feet, as with all the rods, the target is clamped at the $6\frac{1}{2}$-foot division. The back of the rod slides upon the front half and when so extended the vernier is on each of the narrow sides impressed in the wood (See Fig. 36).

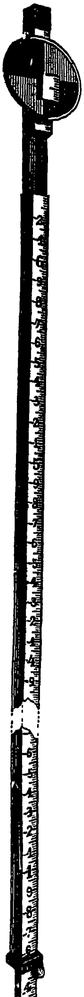


Fig. 34.

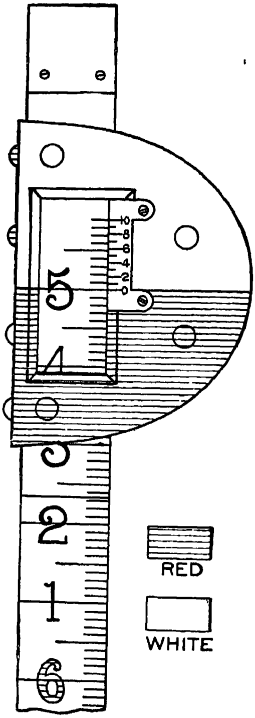


Fig. 35.

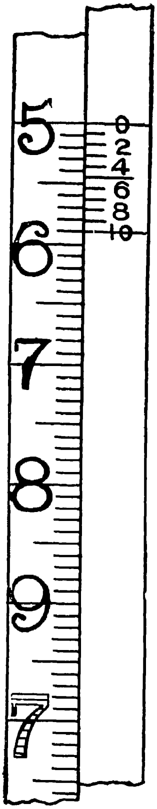


Fig. 36.

back when extended, with the New York rod it is on its narrow sides. The vernier in question is somewhat different from the one found on the Philadelphia rod, since it is provided with a direct vernier, while the other is provided with an indirect vernier. The former, as has been explained, is usually placed below the center of the target, that is, the zero is placed below the intersection of the horizontal and vertical lines of the target. In almost every case this causes confusion because the rodman has been taught, by reason of using a Philadelphia target rod, to read the scale at the zero of the vernier to the fractional parts of a foot by looking along the vernier for the coinciding lines. There need be very little confusion in reading the New York rod, if it is remembered that the center of the target is set by the leveler and not the zero of the vernier as on the Philadelphia rod. Careful observation of the vernier will show that the zero of the vernier is placed at the intersection of the horizontal and vertical lines of the target. The method of using the clamps, setting the target, etc., is the same as that of the Philadelphia rod.

The Boston Rod is made of mahogany, is of the same length and slides out as the rods just described. It is distinctly a target rod and cannot be read without its aid. The scale and vernier, however, are on the narrow sides and can be read to thousandths or any fractional part of a foot. The target is fixed upon one-half of the rod for elevations less than 6½ feet. The target end is held upon the ground and the front of the rod slides upon the back, as shown in Fig. 37. Above 6½ feet the rod is inverted as shown in Fig. 38, and is then used in much the same way as the New York rod. The figures above referred to show the sides of the rod with its scale upon it. The screws at each end act as clamps. In the old style of Boston rod a wooden target was screwed to the rod with the result that the target would warp and twist or be knocked off, thus rendering the rod useless. The best type of Boston rod is fitted with a bent metal target. This form is very serviceable and satisfactory. It will be apparent from the foregoing description that the rod is read altogether by vernier, the scales and vernier being on the side. It is the lightest and neatest rod of the three but the least white, with black graduations; it is divided into feet, tenths and hundredths, (See Fig. 39). The scale is on both sides. At each end is a spirit level bubble with graduations on the upper side of the tube to bring the rod in a horizontal plane. In the center of the rod is an opening for the hand, and thereby it can be easily taken from place to place. The purpose of this rod is to simplify the lengthy calculations in taking cross sections; this will be more fully explained under its respective head.

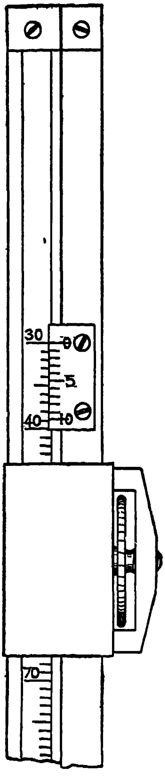


Fig. 37.

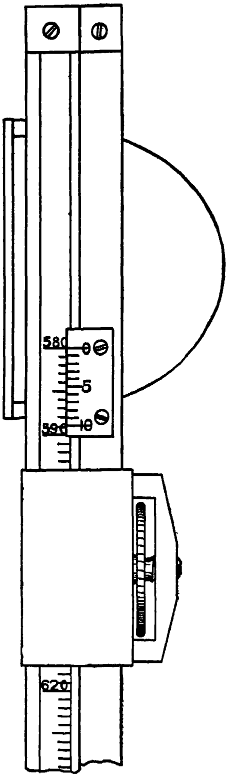


Fig. 38.

In Fig. 39 A and B are the bubbles. It is apparent that if one end of the rod is placed on the side of a hill and the other raised in a vertical position until the bubble appears in the center of the tube, the base of the rod will be in a horizontal plane.

Ranging Poles. Fig. 40 shows the three forms of ranging poles (called flags) in common use, all of which are from 6 to 10 feet in length, made of hardwood, octagonal in shape; they are tapered from the top down and each foot painted alternately red and white, and provided with steel shoes, except the smallest one, which consists of an iron tubular rod $\frac{7}{8}$-inches in diameter and used for the most part on construction work. These flags are for the purpose of establishing points or retaining a given line indefinitely; it is an important tool of the surveyor's outfit. To use this flag, it is placed approximately at some reasonable distance from the instrument and then by the signals of the observer is moved until the line of sight through the instrument bisects it. It should be held in a vertical position and governed position. It is also a convenient device for measuring, approximately, distances not exceeding six feet, but where any great amount of accuracy is desired, the method should not be relied upon. It is an advantage, however, to use the flag as a check, when it may appear that some discrepancy has occurred.

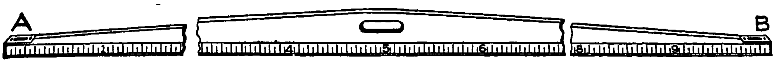


Fig. 39.

INSTRUMENTS.

The Wye Level. There are three kinds of leveling instruments in common use, viz: The Wye level with four leveling screws, the Wye level with three leveling screws and the Dumpy level. The Wye level derives its name from the vertical forked arms, called Wyes, in which the telescope rests. It is clamped to them by collars which may be raised allowing the telescope to be turned on its horizontal axis or lifted out entirely. It is also referred to as the four-screw level. Like other levels it is used for the purpose of ascertaining a horizontal line of sight parallel to a spirit level and perpendicular to the vertical axis. The line of sight is fixed in the telescope by the intersection of cross-hairs. A spirit level is attached to the under side of the telescope and is protected except on top by a metal tube. In the barrel of the telescope slide two tubes, in one of which is an eye-piece; in the other is the objective.

The *eye-piece* usually found with the four-screw leveling instrument is of the erecting type. The inverting eye-piece as distinguished from the erecting eye-piece has two lenses instead of four. The result is that the inverting eye-piece permits more light to reach the eye of the observer, and is therefore better adapted to precise leveling. At first some inconvenience is experienced by the fact that all objects are

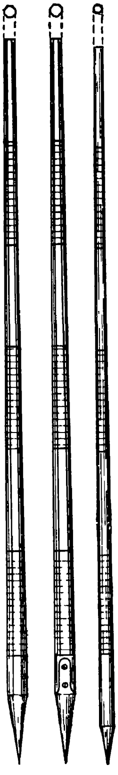
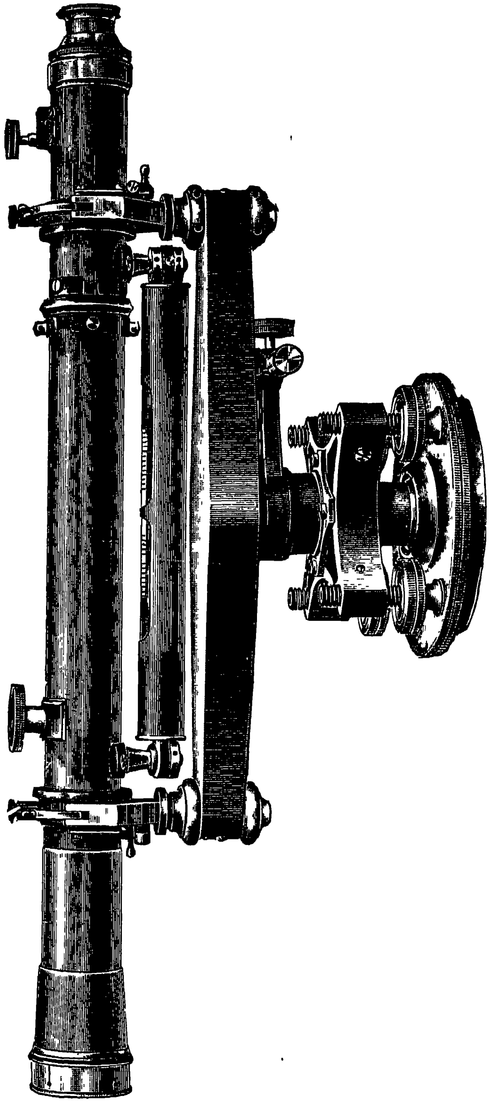


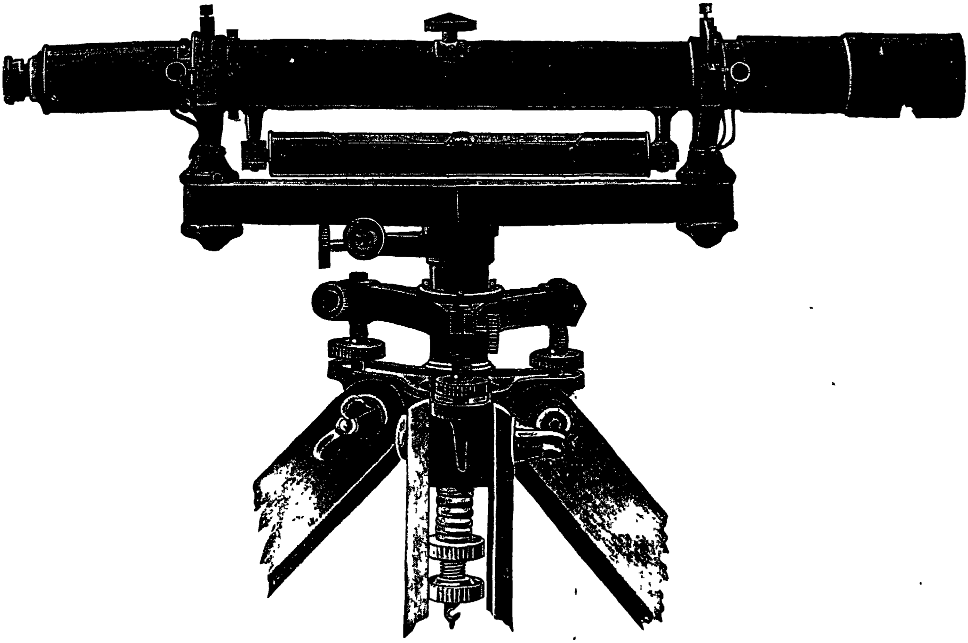
Fig. 40.

screw instrument or a three-screw instrument, lie with the inverting eye-piece. Nearly all makers give a purchaser his choice of the style of eye-piece without extra charge. In purchasing an

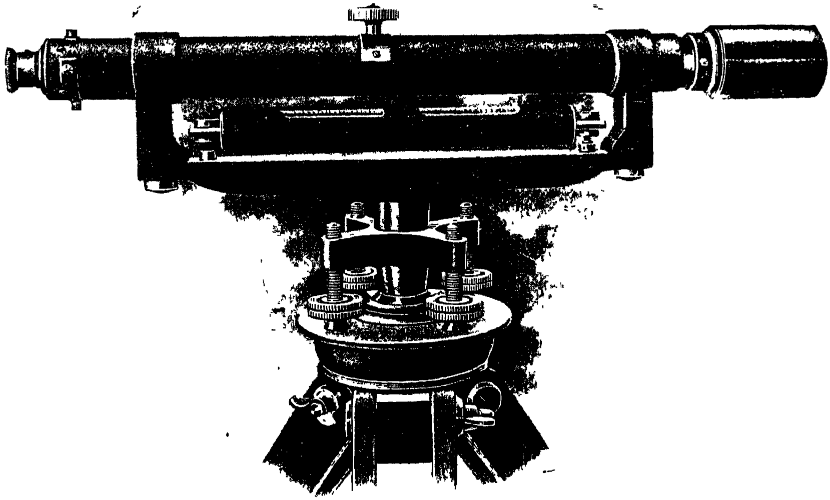


WYE LEVEL—FOUR SCREW.

instrument it should be noticed whether the eye-piece is adjusted by a straight pull or by a spiral motion, because the spiral motion



WYE LEVEL—THREE SCREW.



An inexperienced observer upon looking through the level and finding no cross-hairs may suspect that the cross-hairs are broken. It must not be forgotten that the eye-piece must be focused before the cross-hairs will come into the range of vision. Having once focused the eye-piece upon the cross-hairs, the adjustment will stand for a long time if the eye-piece is undisturbed.

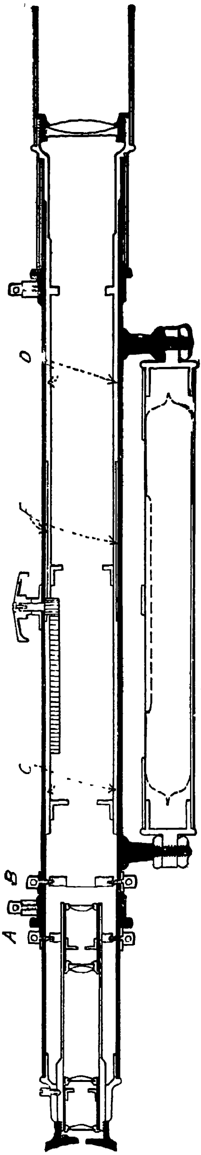


Fig. 42. Section, 18-inch Wye Level.

The object glass is moved in and out by means of a pinion which works on a rack attached to a sliding tube and moves in the axis of the barrel, passing through the run which is inclined in the barrel. The instrument is provided with a clamp, slow motion and leveling screws and mounted on a tripod. The two former screws are situated directly under the horizontal bar and revolve with the telescope.

The Line of Collimation of a level is the line joining the optical center of the object-glass and the intersection of the cross-hairs, and since this line determines the point towards which the telescope is directed, it should coincide with the optical axis of the telescope. The eye-piece and object-glass must be accurately centered.

Instrumental Parallax is an important condition of focusing due to the fact that the image does not fall in the plane of the cross-hairs.

To determine this, direct the telescope upon an object and focus the eye-piece so that the cross-hairs are perfectly distinct. Then turn the telescope upon the object which is to be observed, and focus the object glass until the image is clearly defined. Move the eye from side to side and note whether there is any apparent movement of the cross-hairs and image. If any is seen, the two operations are to be repeated until all parallax is removed.

This adjustment depends upon the eye of the observer and

Spherical Aberration. This defect is caused by combining lenses of different curvatures so that objects on the side of a field of view are seen less distinctly than those in the center. To test the object glass for this defect cover the outer edge with an annular ring of paper and focus upon some desired object. Then remove the ring and cover the central spot of the glass; if no change of focus is needed the glass has no spherical aberration.

To test the eye-piece, sight to a heavy black line drawn on white paper and held near the side of the field of view. If it appears perfectly straight the eye-glass is a good one.

Chromatic Aberration is a defect caused by combining lenses of different and improper varieties of glass so that the yellow or purple colors appear on the edge of the field. To test the telescope for this defect focus it upon a bright distant spot and slowly move the object glass out and in. If no colors are observed around the edge of the field of view the telescope is free from this defect.

Adjustments. The adjustments of the Wye-level are three in number and should be made in the following order:

1. To make the line of collimation parallel to the bottoms of the collars.

2. To make the axis of the bubble tube parallel to the line of collimation.

3. To make the axis of the bubble tube perpendicular to the vertical axis of the instrument.

To make the test for the first adjustment set up the instrument firmly upon solid ground, shaded from sun and wind. Direct the telescope towards the side of a building, a fence or other convenient object and carefully center the intersection of the cross-hairs upon a well-defined point, such as the head of a tack. Clamp the vertical axis and loosen the telescope clips. Now slowly revolve the telescope in the wyes and note if the intersection of the cross-hairs continues to cover the point. If so, the line of collimation is in adjustment.

If the intersection of the cross-hairs moves off the point, revolve the telescope in the wyes as nearly as possible through 180 degrees and carefully center a point at the intersection of the two points and establish a third point: by means of the screws attached to the cross-hair diaphragm, move the diaphragm so that the intersection of the cross-hairs covers the third point. Now repeat the test and correct the position of the cross-hair diaphragm until the intersection of the cross-hairs covers one point as the telescope is revolved in the wyes.

The horizontal cross-hair should at all points be at the same distance from the bottoms of the collars. To test this, carefully center one extremity of the hair upon a point, and by means of the tangent screw, slowly revolve the telescope upon the vertical axis, and if the hair covers the point from end to end, the adjustment is complete. If it does not, the hair is to be adjusted by the same screws as before. Making this adjustment will probably disturb the former one, and the two are to be repeated in succession until satisfactory.

It will be noticed that to make this adjustment, it is not necessary to level the instrument.

Adjustment of the Axis of the Bubble-tube. To test this adjustment, first throw back the clips holding the telescope in the wyes, and then revolve the telescope upon the vertical axis to bring it directly over a pair of leveling screws and clamp the axis firmly. By means of these leveling screws bring the bubble to the center of the tube as accurately as possible. Now without disturbing the instrument, carefully lift the telescope out of the wyes and turn it end for end, being careful when replaced in the wyes that the telescope comes to its seat at each end. If the bubble in this new position of the telescope comes to rest at the center of the tube, the axis of the tube is in adjustment.

If the bubble does not return to the center, bring it one-half way back to the center by means of the vertical screws at one end of the bubble-tube, and the remainder of the way by the two leveling screws. Now repeat the test and correction as often as necessary until the bubble remains in the center of the tube.

Having adjusted the bubble-tube over one pair of screws, test it over he other pair.

When the horizontal hair is truly horizontal, the line of collimation and the axis of the bubble-tube should be in the same vertical plane. To test this been completed, loosen the clips of the wyes and bring the bubble carefully to the center of the tube. Now slowly revolve the telescope in the wyes and note if the bubble still remains in the center of the tube. If it does, the line of collimation and the axis of the tube are in the same plane. If the bubble runs to one end of the tube, bring it back by the horizontal screws attached to one end of the tube.

It must be borne in mind that the first and second adjustments must be carefully made and that they are absolutely essential if satisfactory results are to be attained with the instrument.

Adjustment of the Vertical Axis. This adjustment is not absolutely essential, provided that every time a reading is taken the bubble is brought to the center of the tube by means of the parallel plate screws. However, the adjustment will expedite field work and should always be made.

To test, level the instrument carefully over both pairs of screws; if the bubble remains in the center of the tube as the telescope is revolved on the vertical axis all the way round, the adjustment is complete. If the bubble runs to one end as the telescope is thus revolved, the vertical axis is out of adjustment and may be corrected as follows. Bring the telescope directly over a pair of opposite plate screws, and by means of these screws bring the bubble accurately to the center of the tube. Now revolve the telescope on the vertical axis as nearly as possible through 180 degrees and note the displacement of the bubble: bring the bubble one-half of the way back to the center by the screws, at one end of the bubble-bar attached to the wyes, and the remainder of the distance by the parallel plate screws. Repeat the test and adjustment until the bubble remains in the center of the tube in all positions of the telescope.

Replacing the Cross-hairs. The cross-hairs in leveling instruments may be either spider webs or platinum wire. Spider webs are better, but in selecting them care should be taken to see that they are free from dust and dampness. Probably the web of the little black spider is the most satisfactory, but the web of the spun. Spider webs can best be carried by winding them around a stick.

The cross-hairs are attached to the small diaphragm set at the principal focus of the object-glass. To replace the cross-hairs, carefully remove the diaphragm from the telescope tube and lay it upon a white surface with the cross-hair side upwards. It will be noticed that there are incisions upon the face of the diaphragm intended to indicate the position of the cross-hairs. Fasten one end of the spider web to the diaphragm by means of beeswax or paraffine, carefully suspending the other end of the web over the opposite incision upon the diaphragm; after the web has been properly stretched by fastening it to a match, fasten the web down as before. Repeat the operation with the other cross-hair, then replace the diaphragm in the instrument, care being taken not to break the hairs. The first adjustment may then be made.

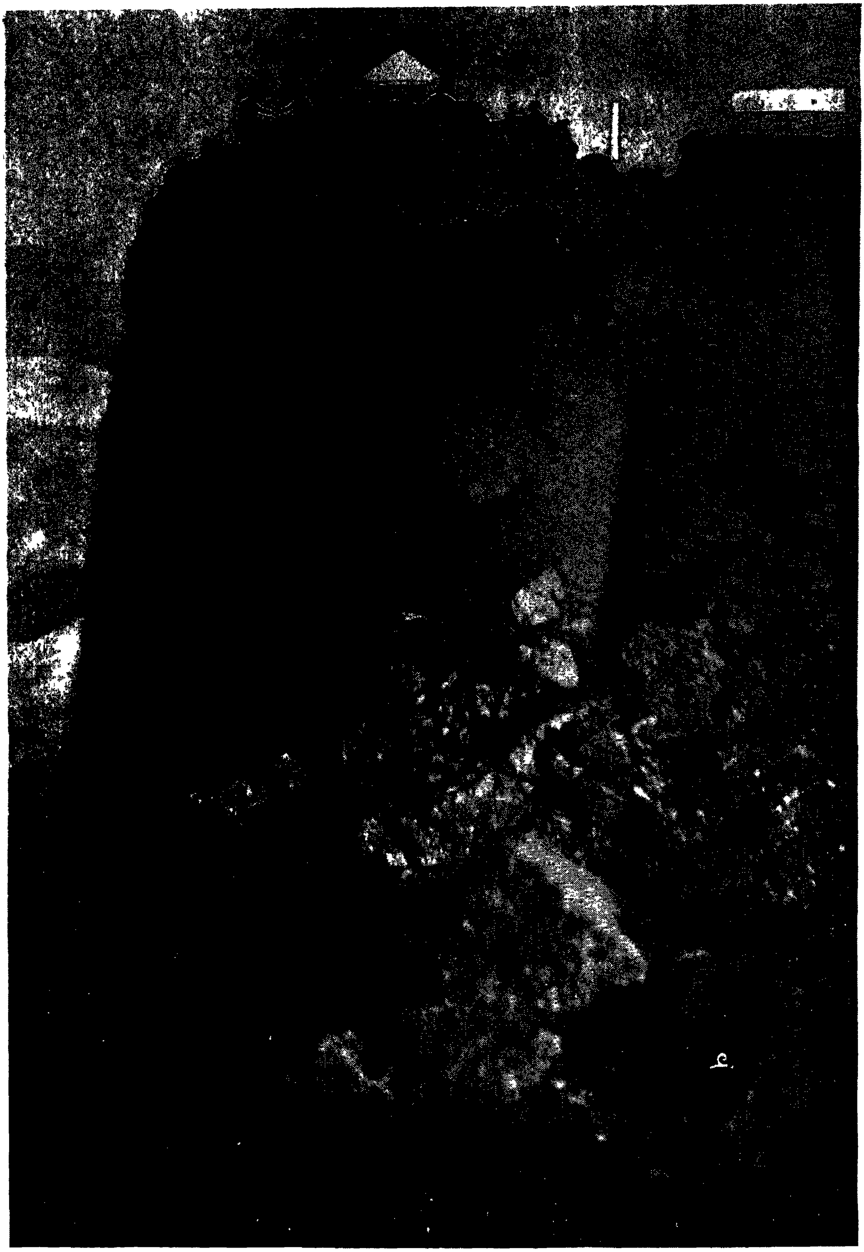
The Dumpy-level. This instrument (see Fig. 41) differs from the Wye-level in the following points: the uprights which carry the telescope are firmly attached to the bar, and the level-tube is mounted on top of the bar. The telescope is attached firmly to the uprights so that the whole structure is rigid. Attached to one of the uprights is a projecting piece with which one end of the level is connected in such a way as to permit of a slight horizontal movement. The other end of the level-tube is fixed with two capstan-headed nuts to permit of vertical adjustment.

A clamp and slow-motion screw should be attached to the center. This, while not an absolute necessity, will, when clamped, prevent unnecessary wear on the center when the instrument is carried upon the shoulder. The telescope may be either erecting or inverting, but the latter is to be preferred.

The dumpy-level, though not as convenient in its adjustments as the Wye-level, will, nevertheless, when properly adjusted, give equally good results; while on account of its simplicity and compactness, it is not so liable to have its adjustments disturbed.

Adjustments. The adjustments of the dumpy-level are two in number and should be made in the following order:

1. To make the axis of the bubble-tube perpendicular to the vertical axis of the instrument



TRIANGULATION STATION ON MOUNT UNCOMPAHGRE. ELEVATION 14,300 FEET

Courtesy of U S Coast and Geodetic Survey, Washington, D C.

2. To make the line of collimation perpendicular to the vertical axis of the instrument.

To make the first adjustment, set up the instrument firmly in a position shaded from sun and wind. Turn the telescope over a pair of opposite plate screws, and by means of these screws bring the bubble accurately to the center of the tube. Repeat the operation over the other pair of screws, and so on alternately over each pair of screws, until the bubble remains as nearly as possible in the center of the tube for both positions of the telescope, care being taken not to swing the telescope through more than 90 degrees.

Now turn the telescope accurately over a pair of opposite plate screws and after leveling carefully, swing the telescope through 180 degrees directly over the same pair of screws. If the bubble remains in the center of the tube, the adjustment is complete. If the bubble does not remain in the center of the tube, bring it one-half way back to the center by means of the vertical capstan-headed screws at one end of the tube, and the remainder of the distance by the leveling screws. Now repeat the test and adjustment until the bubble remains in the center of the tube through all positions of the telescope.

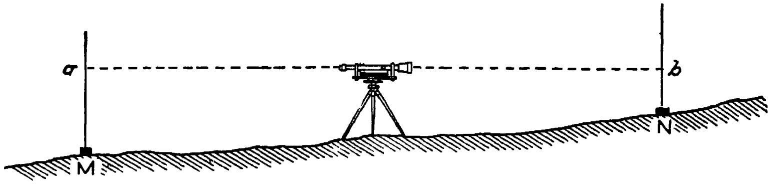


Fig. 43.

The second adjustment of the dumpy-level must be made by the "Peg Method". Select a piece of ground as nearly level as possible and lay out a straight line upon it, from 400 to 600 feet in length, driving a stake at each end and at the center. Set up the level over the center stake and after leveling carefully direct the telescope to the rod held upon N (see Fig. 43), and take the reading by the target. Now direct the telescope to the rod held at M and take the reading. The difference of these two rod readings at N and M will join the two differences of elevations, no

Now set up the instrument within about twenty feet of the stakes, as N, and take the rod reading; carry the rod to M and take the reading. If the difference of these last two readings is the same as before, the line of collimation is in adjustment; if not, correct nearly the whole error by means of the upper and lower capstan-headed screws attached to the diaphragm carrying the cross-hairs. Repeat the test and correction several times until the difference of elevation from both positions of the instrument agree.

It may be well to note right here that the second adjustment of the Wye-level may be made by the "Peg Method," but it is thought that the method given is the more convenient.

The Precise Spirit Level. The description of this instrument which is shown in Fig. 44 is taken from the catalogue of the makers, F. E. Brandis, Sons, & Co.

The principle underlying the construction of the instrument is that the telescope can be moved in a vertical plane about a horizontal axis by means of a micrometer screw. This construction is especially adapted to the object in view, viz: of multiplying the pointings on a mark either in the horizon of the instrument or at an angle above or below.

The superstructure consists of two uprights joined somewhat below their middle by a horizontal plate. The upper portions of the uprights are fashioned into Y's, and carry the telescope and the striding level; the lower portions are cut out so as to leave guide pieces passing outside the lower plate. A capstan-headed pivot screw passes through each guide piece at one end into small sockets in the fixed plate. Passing through the fixed plate, the micrometer screw moves between the guide pieces at the other end and abuts against a small steel surface. The fixed plate carries an index, and one of the guide pieces a corresponding scale to register the whole terms of the micrometer, and also a pointer for reading the subdivisions of the micrometer head of which there are 100.

The Gurley Binocular. The binocular hand level, as the word implies, is a hand level with a double telescope attached. It is similar to the monocular hand level in many respects, except it is provided with screw centering and focusing adjustment and can be adjusted to the different widths of the eye, avoiding all strain to the ocular muscles.

Adjustment. Follow the principles as laid down concerning the Locke hand level.

The Gurley Monocular Hand Level. This instrument is a telescope hand level, by which readings are more definitely determined on a rod at some distance than is possible with the ordinary hand level.

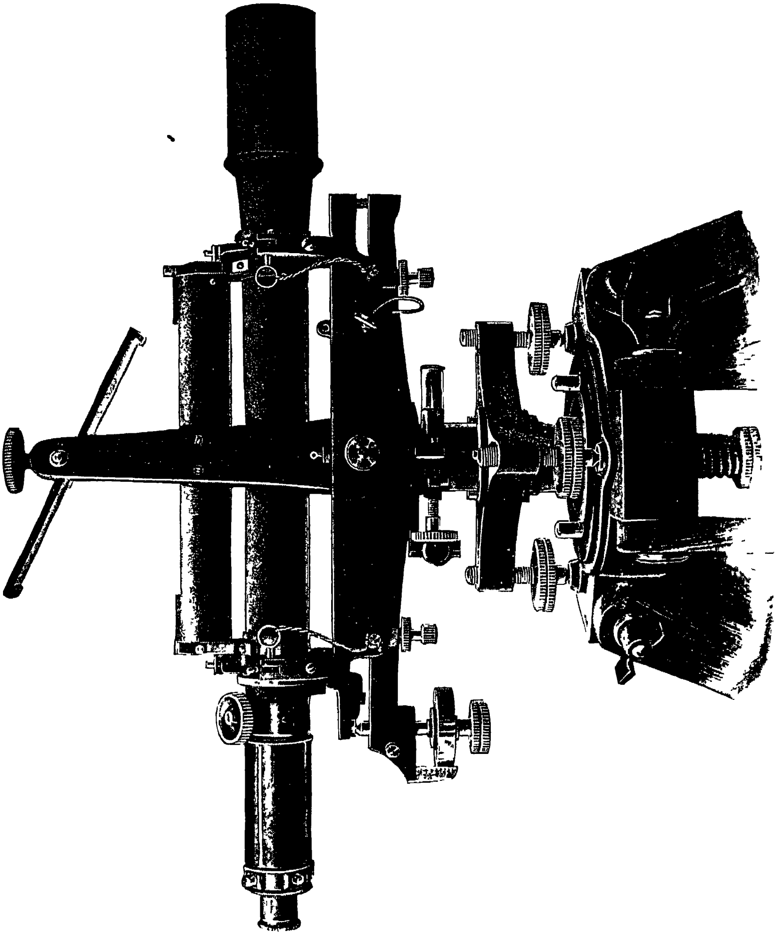


Fig. 44. Precise Spirit Level.

Adjustment. Follow the principles as laid down relative to the Locke hand level.

"Setting up" the Instrument. The term "setting up the level" means to place it in position to secure horizontal sights. To equal distances apart so as to make the leveling plate horizontal. Bring the telescope directly over and in line with the two leveling screws between the plates and opposite each other. As you stand facing the instrument turn the thumb of the left hand in the direction of the motion of the bubble and turn both thumb screws towards or away from each other, being careful not to strain the level plates by having the leveling screws too tight. These screws bear firmly upon the plates and should move ase and smoothness, but there should be no movement

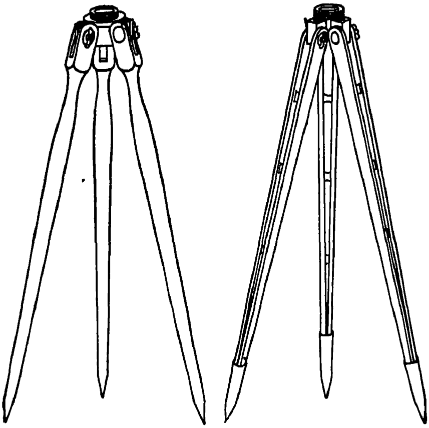


Fig. 45.

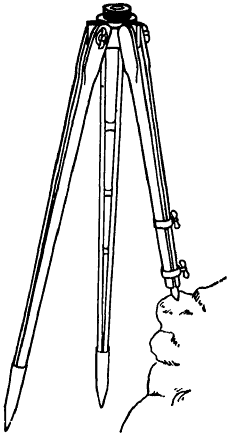


Fig. 46.

of the vertical axis of the instrument. Turn the screws until the bubble appears along the graduations of the bubble tube and bring it to the middle, then turn the telescope at right angles to these two screws and over the other two. In like manner perform the same operation as before. This will cause the bubble to run away from its former position; bring the bubble accurately in the center of the tube over these screws, that is, having equal spaces on each side of the zero of the scale, then turn the telescope over the former screws and bring the bubble in the center. Do this several times until the bubble remains stationary at any angle the telescope may turn; to test this, turn the telescope half way around and see if the bubble moves; should it remain stationary, the instrument is level. The level is, with few exceptions, never placed in line (except when being adjusted under the peg method). It is usually placed in some convenient spot where the greatest number of horizontal sights can be secured. As already stated, the tripod legs must be so placed as to make the plates horizontal. This will save time in bringing the bubble in its proper position. Should it be required to set up the instrument on the side of a hill, place one leg at an altitude and the other two in apparent line with each other (see Figs. 45 and 46), but where the tripod is adjustable the proper method is apparent.

After the instrument is set up and leveled, focus the eyepiece upon the wires and focus the object-glass on the rod by means of the screw placed for that purpose on the top or side of the telescope Care should be taken not to take a reading until the bubble has been carefully observed and brought in the exact center of the bubble tube. When this is completed, sight through the telescope and note the rod reading or set the target rod; again look at the bubble and see if it has moved away from its former position; if not, again sight on the rod and see if the first observation was correct. Should the intersection of the cross-hairs fail to coincide with the horizontal and vertical lines of the target or the center of the rod, the rodman is to incline the rod by the signals of the observer, until it coincides or is in line of collimation.

Care of the Instrument. This duty properly belongs to the instrument man or leveler, and the requirements should be thoroughly understood. While in the field, the instrument remains on the tripod and is carried from place to place as the work requires, but when taken any distance, such as on railway trains, street cars, etc., it should be carefully placed in the box and carried by one who is capable of giving it proper care and attention.

The instrument man being responsible for the instrument, it is natural, and perhaps best, that he should always carry the instrument. In fact, the greatest amount of precaution should be exercised in the care of the instrument, both in the field and while conveying it.

Instruments in general, the level in particular, should never be unduly exposed to the rays of the sun, as this will have a tendency to throw its various sensitive parts out of adjustment, therefore whenever possible, place the instrument, whether it is on the

The leveler should always exercise great care not to disturb the instrument after it is set up and should avoid, as much as possible, walking around it unreasonably, especially if the ground is soft, or the position of the instrument not very firm. This applies to all persons whether in the active performance of duty or not. It is frequently necessary to set up the instrument in places such as loose timber, rocks, etc., thus the importance of this care is apparent. If disturbed to any great extent it will be necessary to relevel it, and if the position of the legs of the tripod is disturbed, the entire work must be done over, because the height of the instrument will not be the same as in its former position. Should the instrument be disturbed after a turning point has been established and its elevation ascertained, it will only be necessary to take a reading on the last turning point to determine the new height of the instrument. After leveling, the instrument man should keep his hands off the instrument except for the purpose of leveling and adjusting the telescope. He should not make a practice of leaning his weight on the tripod. It is often necessary to send instruments great distances, and in so doing, in no case should it be sent by express or freight without first being properly packed and secured against breakage; because of its fine construction and sensitiveness it may get out of adjustment to such an extent as to render it impossible to readjust it for good work by any method known to the engineer, and may become worthless and beyond repair even to an instrument maker.

The student should appreciate that the care of the instrument is just as important to good work as its original excellence.

Leveling. To determine the difference in elevation between two points, both of which are visible from a single position of the instrument, set up the instrument in such a position that the rod held upon either point will be visible. Now send the rod to one of the points as at A in Fig. 48; direct the telescope upon it and take the rod reading; now direct the telescope to the rod held at B and again note the reading. Evidently the difference of the rod will give the difference in elevation of the two points.

If the points are too far apart or if the difference of elevation is too great to be determined from one setting of the instrument, desired to find the difference of elevation of A and C in Fig 47, C being too far below A to permit of being read upon both points from a single position of the instrument. Set up the instrument (not necessarily on line from A to C) in some position such that the line of sight will strike the rod as near its foot as it is possible to take a reading: send the rod to some point B such that the line of sight will strike the rod near the top when extended. The difference of these rod readings will give the difference of level of A and B. Now carry the instrument to some point such that rod readings can be taken upon B and C. The difference of the rod readings upon B and C added to the difference of rod readings upon A and B will give the difference of level of A and C, proper attention being given to signs.

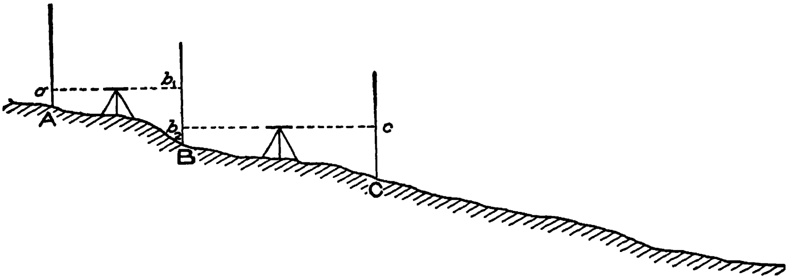


Fig. 47.

If the line of levels is very extended the above method is awkward, as some of the differences will be positive and some negative. Choose some plane called a datum plane, such that all of the points in the line of levels will lie above it.

Beginning at the point A, assume the elevation of the point above the datum plane. Read the rod held upon A, and the reading added to the assumed elevation will give the height of the cross-hairs above the datum plane, called the "height of instrument" (H.I.). Now, turn the instrument upon the point B and read the rod and it is evident that this last rod reading subtracted from the height of instrument will give the elevation of B above the datum plane. Next move the instrument beyond B, or at least where it can command a view of B and C and again sight to the rod held upon B. This last rod reading added to the elevation reading at C is subtracted will give the elevation of C above the datum plane. Fig. 48 will make the method of procedure apparent.

Referring now to that figure, the first rod reading taken upon the point A is ordinarily called a "back-sight" and the first reading taken upon B is called a "fore-sight". There seems to be no good reason for adhering to this method of distinguishing between the rod readings and it is illogical and misleading. A back sight is not necessarily taken behind the instrument, that is, in a direction contrary to the progress of the survey, neither is a fore-sight necessarily taken in front of the instrument. It is more logical and less misleading to designate these rod readings by the terms "plus-sight" and "minus-sight".

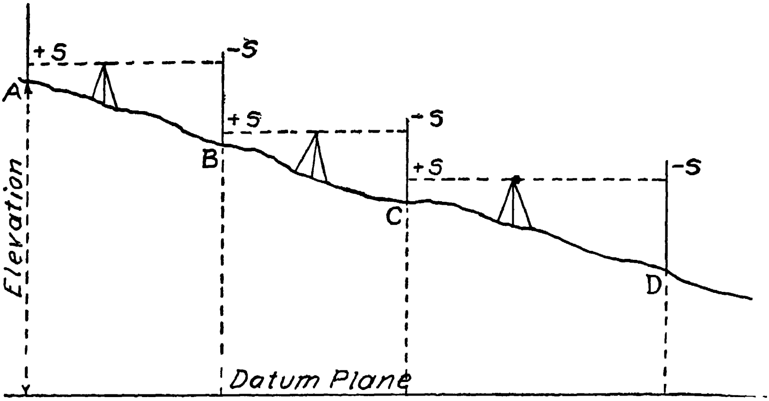


Fig. 48.

A plus-sight, therefore, is one taken upon a point of known or assumed elevation, to determine the height of instrument.

A minus-sight is one taken upon a point of unknown elevation and which, subtracted from the height of instrument, will give the required elevation.

A "bench-mark" (B.M.) is some object of a permanent character, the elevation of which, together with its location, is accurately determined for future reference and for checking the levels.

A "peg", "hub", or turning point" (T.P.), is a point used for the purpose of changing the position of the instrument. This turning point may be taken upon a bench-mark, but is oftener taken upon the top of a spike or stake driven into the ground.

If a self-reading rod is used, the instrument man will carry the notebook and record the rod readings as they are observed. The leveler should cultivate the practice of calculating the elevations of his stations as the work progresses, thereby enabling him to discern errors when they occur.

If a target rod is used upon the work, the rodman should also carry a notebook in which he should at least enter all readings upon turning points and bench-marks and check up with the instrument man at every opportunity. Under the circumstances, the instrument man is more or less dependent upon his rodman for the correct reading of the rod and when an inexperienced rodman must be employed, the self-reading rod will give the better results.

The limit of range of an ordinary leveling instrument is about 400 feet, and sights should not be taken at a greater distance.

The method of keeping the field notes for the work above outlined is given below. A level notebook especially adapted to the purpose should be procured, the notes entered on the left-hand pages, the right-hand pages being reserved for remarks, sketches, etc.

STA.	+ S	H. I.	—S	ELEV.	
A	0.650	1000.650		1000.00	⊗ N. E. cor. of abutment Main street bridge.
B	1.250	993.140	8.760	991.890	
C	2.380	987.670	7.850	985.290	
D			9.570	978.100	

It will be noticed that the algebraic sum of the plus and minus readings equals the difference of elevation of the first and last stations, and these quantities should be checked as often as possible to discover errors in addition or subtraction.

Profile Leveling. The method of profile leveling is the same in principle as above outlined, but the details of field work are a little different.

In this sort of work it is intended to determine a vertical section of the ground above a datum plane. To this end, rod read-are plotted and the points connected, the resulting irregular line will closely approximate the actual line of the surface.

Profile levels are usually run in connection with a transit or chain survey of the line, the positions of the points being first established upon railroad surveys. These points are usually 100 feet apart unless the ground is very irregular, when they may be 50 or 25 feet apart or even less, the points being indicated by stakes. Upon sewer or street work they should seldom be more than 50 feet apart and the readings should be taken with the rod held upon the ground.

Fig. 49 will illustrate the difference between profile leveling and the first system outlined, sometimes called differential leveling or "peg" leveling. Referring now to that figure A B is the datum

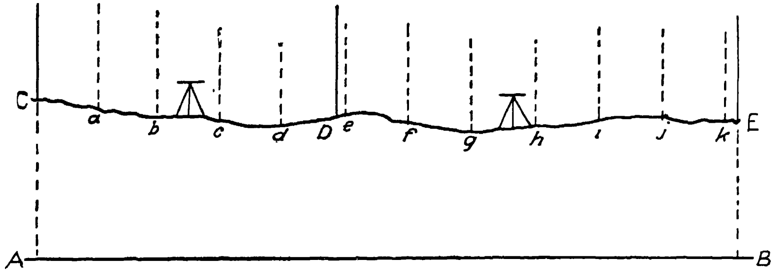


Fig. 49.

plane and the full lines at C, D, and E represent positions of the rod for turning points. Assuming the elevation of the point C, the rod is held upon it and the reading added to the elevation for the height of instrument. The rod is then carried successively to the points *a*, *b*, *c*, *d*, and each reading is in turn subtracted from the height of instrument at C, to get the elevations of these points.

The rod is then held upon the point D, the instrument moved and the plus-sight upon D added to the elevation for the new height of instrument. The rod readings upon *e*, *f*, *g*, *h*, *i*, etc., are then each subtracted from this new height of instrument for elevations.

This figure illustrates the improper use of the terms back-sight and fore sight. The rod readings at C and D are taken behind the instrument, but the rod reading at C is the only plus-sight.

The method of keeping the field notes is illustrated below.

Sta.	+S	H. I.	—S	Elev.
O.	3.25	585.70		582.45
+ 50			3.78	581.92
1			4.18	581.52
+ 50			5.06	580.64
T. P. + 62	2.64	581.49	6.85	578.85
2			3.10	578.39
+ 50			3.18	578.31
3			3.90	577.59
+ 50			4.60	576.89
4			5.25	576.24

CROSS-SECTIONING.

One of the most important problems that confronts the leveler is the setting of "slope stakes," called cross-sectioning, from which may be determined the amount of earthwork in cut or fill, and which mark the extreme limits of the operations of the construction corps in building railways, highways, sewers, canals, irrigation ditches, etc.

The problem is as follows: Given the required width of finished roadbed or channel, with proper side slopes (depending upon the kind of material), it is required to determine where these side slopes will intersect the natural surface of the ground with reference to the center line of the survey. The center line is defined by stake, carefully aligned and leveled, and a profile of it

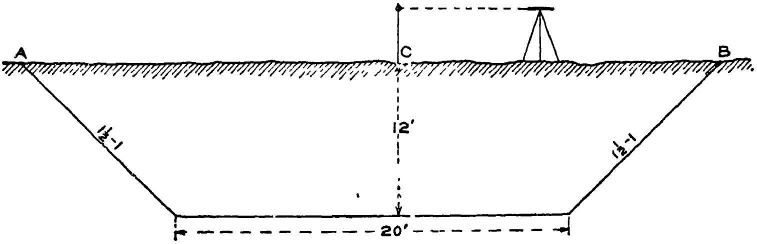


Fig. 50.

elevation of the finished roadbed or channel with reference to the natural surface of the ground.

Let us assume the ground to be level transverse to the center line. Depth of cut at center = 12 feet; side slopes 1½ feet horizontal to 1 foot vertical; width of cut at bottom = 20 feet. See Fig. 50.

Set up the instrument in some convenient position that will command a view of as much ground as possible. Hold the rod upon the ground at the center stake and note the reading. Suppose it to be 3.5 feet. Now if the ground is level, the distance from C to B is evidently $10+(12\times1\frac{1}{2})=28$ feet and the rod should again read 3.5 feet when held at B. The point A would be found in the same way.

The notes would be kept as shown below.

Sta.	Dis.	Left	Center	Right		Area	C.Yds.
175	50	$\frac{+12.0}{28}$	+12.0		$\frac{+12.0}{28}$		
176		$\frac{+\ 3.0}{14.5}$	+ 6.0		$\frac{+\ 9.2}{23.80}$		
176	50	$\frac{+\ 2.5}{13.75}$	+ 5.0	$\frac{+7}{9}$	$\frac{+\ 8.0}{22}$		

The preceding example illustrates one of the simplest cases that occur in practice. Let us now take the case of a line located upon the side of a hill. See Fig. 51.

Depth to grade at center 6 feet; width at bottom 20 feet; side slopes 1½ to 1. As before, hold the rod upon the ground at C and determine the height of instrument above C. Suppose this to be 5.5 feet. Now, if the ground were level through C it would be necessary to measure to the right $10+(6\times1\frac{1}{2})=19$ feet to the point D and the rod should read 5.5 feet. Instead it reads, say 2.8 feet. We know therefore that we have not gone out far enough by (5.5—2.8) 1½ = 4.05 feet, if the ground were level through the point D, bringing us to the point E where the rod should read 2.8 feet. Suppose it reads 2.3 feet. We must then go out 0.75 foot farther, each move bringing us closer and closer to the point B. This operation may be repeated as often as is work the instrument man can direct the rod closely enough to the point B for all practical purposes. We then enter the notes in the second line of the record shown above.

Upon the left of the center, these operations are reversed. That is to say, we measure out 19 feet and instead of the rod reading 5.5 feet, it reads, say, 8.5 feet. We know then that we are out too far by 4.5 feet. We then move in toward the center the required distance and read the rod again, noting how much it differs from 8.5 feet, if any, and enter the final results in the notes.

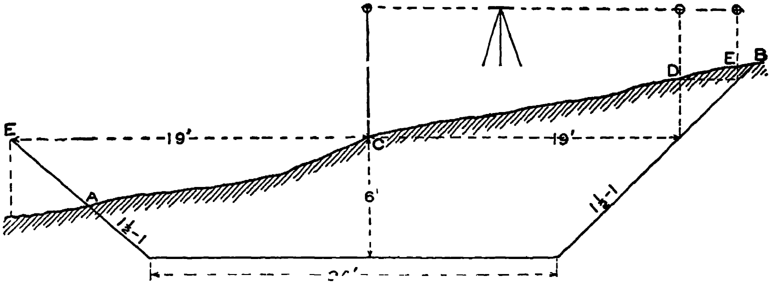


Fig. 51.

A third case is shown in Fig. 52, in which the transverse slope is not uniform. The method of procedure is the same as in the other cases, but the rod should be held at the point where the slope changes in order to find its height above grade. Enter this and the distance out in the third line of the notes.

The transverse section may be very irregular, in which case it may be necessary to take readings at several points in order to calculate the area of the sections with more exactness. At times

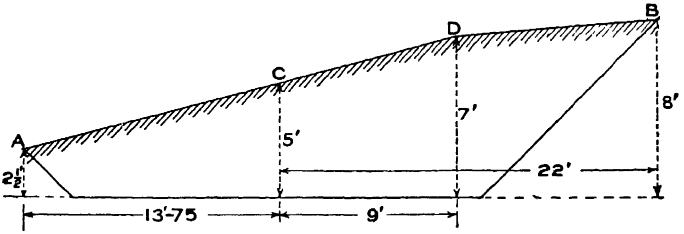


Fig. 52.

compound section. Each material will, of course, have its own proper side slope, and the depth and extent of each must be determined by soundings.

In case the section is in fill instead of in cut, the method is the same as in the preceding cases, as will be illustrated in the following examples. Let us first take a section level transversely. See Fig. 53.

In this case the finished grade is to be 9 feet above the point C. Hold the rod at C and suppose it reads 3.25 feet. Now since the ground is level we go out to the right and left $9+(9\times 1\frac{1}{2}) = 22.5$ feet and set the stakes at A and B entering the record in the notebook as before, except that now the numerator of the fraction will be marked — instead of +.

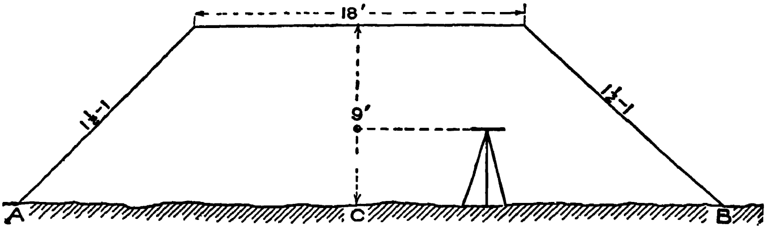


Fig. 53.

We will next take the case where the surface of the ground has a transverse slope. See Fig. 54. Now hold the rod at the point C, and suppose it reads 9.25 feet. Now if the ground were level through C we would have to go out to the right $9+(6.25 \times 1.5) = 18.4$ feet to some point D. But there the rod reads, say, 1.5 feet, hence we know we are out too far by $7.75\times 1.5 = 11.63$

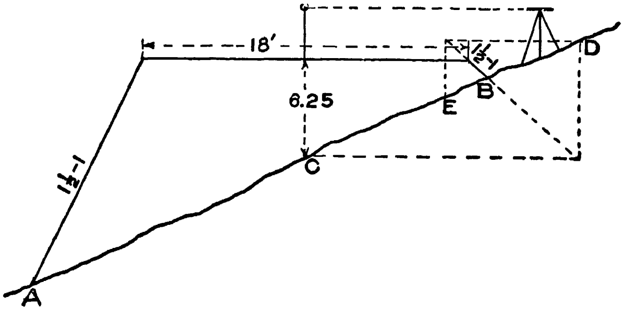


Fig. 54.

say, 3.5 feet and we move out again $2.0 \times 1.5 = 3$ feet. Therefore we move back and forth until we find the point B where the computed rod reading and the actual reading agree.

Sometimes it will be found that a part of the section is in cut and a part in fill, but methods outlined will serve in any case.

The distance between the sections longitudinally will depend upon the nature of the ground. On uniformly sloping or level ground they may be taken 100 feet apart. Over uneven ground it may be necessary to take them as closely together as 25 feet or even less. In the sections themselves, a sufficient number of rod readings should be taken that the area of the sections may be determined with reasonable accuracy.

After the field work is completed, the notes are plotted, usually upon cross-section paper, and the areas determined either with a planimeter, by Simpson's rule or some other method. These sections then divide the earthwork into a system of prismoids of which the volume must be calculated. The formula for calculating volumes is known as the Prismoidal Formula and is as follows:

$$\frac{l}{6 \times 27}(A+4M+B)$$

in which l = length between consecutive sections, A = one end section, B = the other end section and M = the section midway between the two. The result is given in cubic yards.

The mistake must not be made of assuming that M is a *mean* between A and B; but a theoretical section must be plotted whose dimensions are a mean between those of A and B. This often results in quite a complicated problem, and various other formulas have been devised to give sufficiently close results without the labor and time involved in the preceding. This will be treated in detail in Railroad Engineering.

UNITED STATES ENGINEERS SURVEYING IN ALASKA

PLANE SURVEYING

PART II

The **meridian plane** of any place upon the earth's surface is a plane passing through the zenith of the place and the poles of the earth. A true meridian is, therefore, a line lying in this plane, and would, if produced, pass through the poles.

The magnetic meridian plane would in the same way be defined by the zenith and the magnetic pole of the earth; but since this pole is not fixed in position, the magnetic meridian is defined as the direction of the line indicated by the position of the magnetic needle.

At a few places upon the surface of the earth, the true meridian and the magnetic meridian coincide *at times*, but for the most part they differ in direction by an ever varying quantity. The angle at any place between the true meridian and the meridian as defined by the magnetic needle, is called the **magnetic declination** for that place. If the direction of the magnetic meridian were constant, or if the changes followed any particular law, it would be a comparatively simple matter to determine the declination for any time or place. The variations occurring are of three principal kinds—diurnal, annual, and secular, the last being the most important.

Diurnal Variation. On continuing observations of the direction of the needle throughout the day, it will be found that the north end of the needle will move in one direction from about 8 A. M. until shortly after noon, and then gradually return to its former position.

Annual Variation. If observations be continued throughout the year, it will be found that the diurnal changes vary with the seasons, being greater in summer than in winter.

Secular Variation. If accurate observations on the declination of the needle, in the same place, are continued over a number of years, it will be found that there is a continual and comparatively constant increase or decrease of the declination, continuing in the

Besides the above, the declination is subject to variations more or less irregular, due to local conditions, lunar perturbations, sun spots, magnetic storms, etc.

The declination in any part of the United States may be approximately determined by consulting the chart issued from time to time by the United States Coast and Geodetic Survey. (See chart, page 132.) Upon this chart all points at which the needle points to the true north are connected by lines, called **agonic lines** or **lines of no declination.** Lines are also drawn connecting points of the same declination, called **isogonic lines.**

The isogonic curves or lines of equal magnetic declination (variation of compass) are drawn for each degree, a + sign indicating West declination, a — sign indicating East declination.

The magnetic needle will point due North at all places through which the agonic or zero line passes, as indicated on the chart.

Before undertaking an extensive or important survey, it is the first duty of the surveyor to determine accurately his declination. This is best done by laying out a true meridian upon the ground and comparing its direction with that indicated by the needle. Before describing the methods of laying out a true meridian, it will be best to describe the compass.

THE COMPASS.

Construction. The surveyor's compass consists primarily of a circular brass box, carrying, upon a pivot in its center, a strongly magnetized needle (see Fig. 56). The inside edge of the box on a level with the needle, is usually graduated to half degrees, and smaller intervals may be "estimated." Two points diametrically opposite each other are marked 0°, and form the north and south ends of the box, the *south* end being indicated by the letter S, and the *north* end either by the letter N or by a fleur-de-lis or other striking figure. The divisions extend through 90° upon both sides of these points, to the east and west points marked respectively E and W. The east side of the box, however, is on the left as the observer faces the north end; this is because the needle remains stationary while the box revolves around it. The divided circle is sometimes movable, being fitted with a clamp and tangent-screw

The **magnetic needle** is the most essential part of the compass. It consists of a slender bar of steel, usually five or six inches long, strongly magnetized, and balanced on a pivot so that it may turn freely and thus continue to point in the same direction however much the box carrying the pivot may be turned around. To this end the pivot should be of the hardest steel, ground to a very fine point, or, better still, of iridium; and the center of the needle resting upon the pivot should be fitted with a cap of agate or other hard substance.

To distinguish the ends of the needle, the north end is usually cut into a more ornamental form than the south end, or the latter end may be recognized by its carrying a coil of wire to balance the "dip."

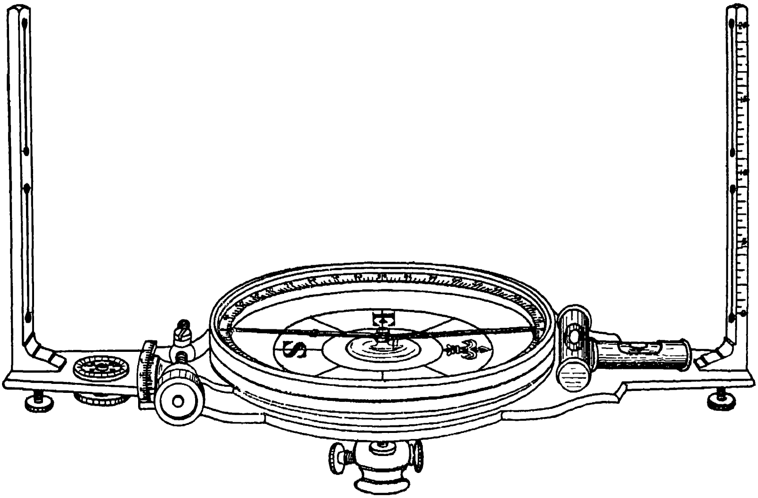


Fig. 56.

Intensity of directive force and sensitiveness are the chief requisites in a magnetic needle, and nothing is gained by making a needle over five inches in length. Indeed, longer needles are liable to have their magnetic properties impaired by polarization. The needle should not come to rest too quickly. Its sensitiveness is indicated by the number of vibrations that it makes in a small space before coming to rest. Should it come to rest quickly or be is weak or that there is undue friction between needle and pivot.

The under side of the box should be fitted with a screw which, engaging a lever upon the inside of the box, will serve to lift the needle off the pivot when the instrument is carried about.

The **sights** form the next most important feature of the compass. They consist of two brass uprights, with a narrow slit in each, terminated at intervals by circular apertures. They are mounted directly upon the compass-box; or the bottom of the box may be extended at each end in the form of a plate, and the sights attached at the ends of the plates. However mounted, the sights should have their slits in line directly over the north and south points of the divided circle. The right and left edges, respectively, of the sights, may have an eye-piece and a series of graduations, by which angles of elevation and depression for a range of about twenty degrees each way can be taken with considerable accuracy. This device is called a **tangent scale,** so called because of the distance of the engraved lines from the 0° line being tangents (with a radius equal to the distance between the sights) of the angles corresponding to the numbers of the lines.

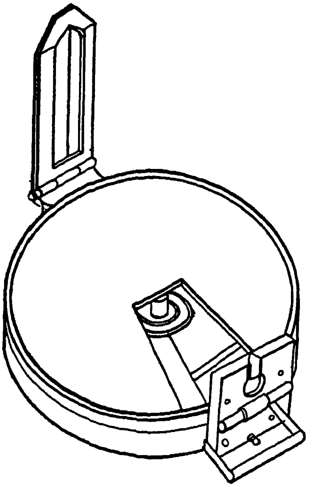


Fig. 57.

The spirit levels may be placed at right angles to each other in the bottom of the compass-box, or mounted in the same way upon the plate.

The compass is usually fitted to a spindle made slightly conical, which has on its lower end a ball turned perfectly spherical, confined in a socket by a pressure so light that the ball can be moved in any direction in leveling the instrument. The ball is placed either in the brass head of a Jacob staff, or, better, in the top casting of a tripod.

A plumb-bob should be provided with the instrument to center it over a stake.

A telescope is sometimes provided, to be attached to one of line of sight. The compass is, however, so inaccurate that it would seem to be an unnecessary refinement.

Prismatic Compass. This is a form of compass used in general where merely ordinary work is required. It is about 3 inches in diameter with a floating metal dial (see Fig. 57), and is provided with folding sights and prism. By means of the latter it may be read while being pointed. This is especially useful when the instrument is held in the hand. Although it can be mounted on a Jacob staff, it is usually held in the hand and carried in the observer's pocket.

Adjustment. *To Adjust the Levels.* First bring the bubbles to the middle of the tube by the pressure of the hand on different parts of the plate, and then turn the instrument half-way round. If the bubbles remain in the middle of the tubes, the tubes are in adjustment. If the bubbles do not remain in the middle, raise or lower one end of the tube to correct one-half the error. Relevel the instrument, again test, and apply the correction as before. Continue the operation until the levels are in perfect adjustment.

To Adjust the Needle to the "Dip." While the compass is still in a perfectly level condition, see if the needle is in a horizontal plane. Should this not be the case, move the small coil of wire towards the high end until the needle swings horizontally.

To Adjust the Sight-Vanes. Observe through the slits a fine hair or thread made exactly vertical by a plummet. Should the hair appear on the side of the slit, the sight-vane must be adjusted by filing its under surface on the side that seems the higher.

To Adjust the Needle. Having the eye nearly in the same plane with the graduated rim of the compass-box, bring one end of the needle in line with any prominent graduation mark in the circle, as, for instance, the zero or the 90-degree mark, and notice if the other end corresponds with the same degree upon the opposite side; if it does, the needle is said to "cut" opposite degrees; if not, bend the center pin, until the ends of the needle are brought into line with the opposite degrees.

Then, holding the needle in the same position, turn the instrument half-way round, and note whether the needle now cuts needle, and the other half by bending the center pin. The operation of testing and correcting should be repeated until perfect reversion is secured in the first position. This being obtained, the operation should be tried on another quarter of the circle; if any error is found, the correction must be made in the center pin only, the needle being already straightened by the previous operation. When the needle is again made to cut, the test should be tried in the other quarters of the circle, and the correction made in the same manner, until the error is entirely removed and the needle will reverse at every point of the graduated circle.

Use. In the operation of locating points, and therefore lines, by angle-measuring instruments, two operations are necessary:—(1) to measure the angle at the instrument between some given line and the line passing through the given point; (2) to measure the distance from the instrument to the given point. For the first operation two types of instrument are in general use—the compass and the transit. For the compass, the line of reference from which all angles are measured is a meridian, and the angular deviation from this line is called the **bearing.** The bearing and length of a line are collectively named the **course.** The compass, therefore, measures bearings directly and angles indirectly.

To Determine the Bearing of One Point from Another. "Set up" the compass over one of the points, and level carefully. Turn the sight-vanes in the direction of the second point, *with the north end of the plate ahead.* Hold a rod upon the second point, and cover it with the slits in the sight-vanes. Now lower the needle upon the pivot, being sure that the instrument is still level; allow it to come to rest, and read the bearing.

To Survey a Series of Lines with the Compass. "Set up" the compass over the point A, with the north end of the plate ahead (Fig. 58); and after leveling, turn the sight-vanes to cover a rod held upon the point B. Now send out the tape in the direction of B, and, sighting through the slits, signal the head tapeman into line. Continue this until the point B is reached. Now read and record the bearing and the length of the line. Take up the instrument, and carry it to B. Set it up over B, with the north end ahead, that is, pointing in the direction of the survey. Level, and

Read the bearing as a check upon the former one, but reversed in direction; *i. e.*, if the bearing from A to B was north by east, the bearing from B to A will be south by west. If the direct and reversed bearings check, turn the north end of the compass to cover a rod held upon C. Read the bearing, measure B C, take the instrument to C, and proceed as before.

If at any station, such as C, the direct and reversed bearings do not agree, take the instrument back to B and again take the bearing of B C. If they still disagree it indicates local attraction at C. Take the instrument to D and take the bearing of D C, comparing it with the bearing of C D. If these disagree, record the bearings of B C and D C as well as those of C B and C D. The latter should check the former, since the local attraction at C will affect both lines equally; and the correct angle between the lines can be computed.

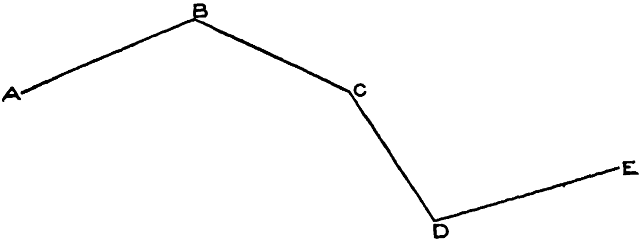


Fig. 58.

Locating a series of lines with certain lengths and bearings is essentially the same as above, except that after the compass has been turned in the proper direction, the stations must be brought into proper line.

Here it may be well to remark upon the proper method of reading and recording bearings. Always read the north or south end of the plate first; *i. e.*, if a line has a bearing 35° east of north, it should be read and recorded N 35° E. If the bearing is 90° east or west of north or south, record the bearing as E or W.

The Gunter's chain is always used in land surveys made with the compass, and deeds and records of such surveys are based upon the Gunter's chain as the unit.

Hints Regarding the Use of the Compass. Sometimes, as etc., the compass cannot be set upon the line. In such a case measure off equal distances at right angles to the line, and find the bearing of the parallel line; the length should be measured upon the line itself. In other cases it may be more convenient to set the compass or rod "in line" upon the line produced, or upon some intermediate point of the line.

It is more important to have the compass level, crosswise of the sights, than parallel with them.

Avoid reading the bearing from the wrong number of the two between which the needle points, as for instance 35° for 25°.

Check the vibrations of the needle by gently raising it off the pivot and lowering it again by means of the screw on the under side of the box.

If the needle is slow in starting, smartly tap the compass to destroy the effect of any possible adhesion to the pivot or friction of dust upon it.

Avoid holding the pins, axe, or any other body of iron, in close proximity to the needle.

Should the needle adhere to the glass after the latter has been dusted with a handkerchief or has been carried so as to rub against the clothes, the trouble is due to the glass being thereby charged with electricity and may be obviated by moistening the finger and applying it to the glass.

RELOCATION.

Suppose it is required to relocate a line, no trace of the old survey being at hand except the given line. Now, between the date of the old survey and the present, the declination of the needle has changed several degrees. The first duty of the surveyor is to consider this question very carefully, and to ascertain the probable amount of change in the magnetic needle. Suppose the result of his inquiry leads to N 38° 15′ E as the bearing. Starting at corner A, Fig. 59, the surveyor runs a random line AS on the bearing N 38° 15′ E, and measures along this line a distance of 32 chains, or 2,112 feet, to point S. On arriving at S, the surveyor proceeds to look over the ground on both sides of this point for a lost corner, which is described in the old record as a monument, no trace of this mark can be found, nothing further can be done from the data at hand. However, should the mark be found at *m*, a perpendicular is dropped upon the line AS, and its length is measured, as is also the distance *n*S. It is now evident that the distance A*n* becomes known. From the right triangle, the angle *n*A*m* can be computed, and the present magnetic bearing of A*m* can be determined.

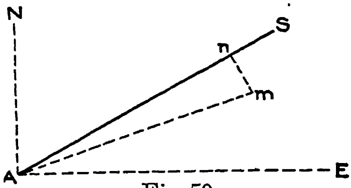


Fig. 59.

For example, suppose that *mn* is found to be 37.4 feet, while A*n* is 2,110.5 feet, then tan. $nAm = \frac{mn}{An} = 0.01772$, whence $nAm = 1° 01'$, and the present magnetic bearing of A*m* is N 39° 16′ E. The distance $Am = \frac{2{,}110.5}{\cos. 1° 01'} = 2{,}110.84$ feet. This indicates that the present work is correct, and that the old survey was in error by 1.16 feet. As there is a principle of law that establishes corners and monuments, resurveys must control; therefore the new record of the line A*m* is N 39° 16′ E, 2,110.84 feet. Intermediate points of the line A*m* may now be established from the starting point A, running it out with the new bearing.

EXAMPLE FOR PRACTICE.

Compute the distance and bearing of two points which are not intervisible. Call the line GH. A line is run approximately near H, from the known corner G to a point A which is visible to H: the bearing and length of this line being N 42° 15′ E, 714.5 feet AH being N 1° 08′ E, 210.5 feet.

Ans. N 33° 14′ E, 883.24 feet.

To Find the Bearing of One Line to Another. Suppose, in Fig. 60, that of the tract of land therein described there has been prepared a rough plot upon which the angles, bearings, and distances as taken from the field book are figured. In order to find the bearing of one line to another, add together the interior angles formed at all the corners; call their sums *a*; multiply the number of the sides by 180°; from the product subtract 360°. If the remainder is equal to *a* this is proof that the angles have been there will always be some discrepancy, but if the field work has been performed with reasonable care the discrepancy will not exceed two minutes for each angle. In this case divide it, in equal parts, among all the angles, adding or subtracting, as the case may be, until it amounts to less than one minute for each angle, when it may be entirely disregarded in common farm surveys.

The corrected angles may now be marked on the plot in ink, and the penciled figures erased. We shall suppose the corrected ones to be as shown in Fig. 60. Next, by means of these corrected angles, correct the bearings also.

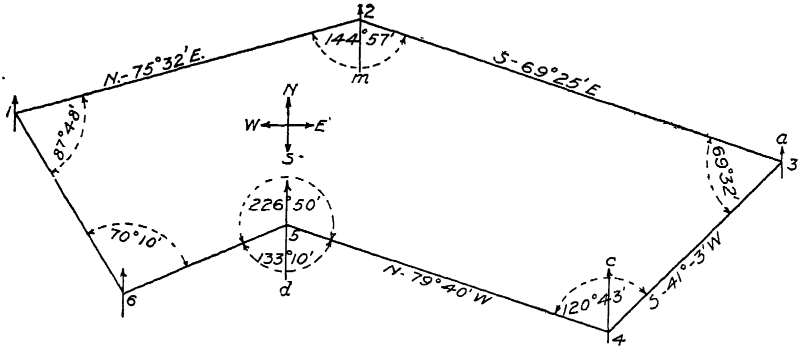


Fig. 60.

Select some side, the longer the better, from two ends of which the bearing and the reverse bearing agree, thus showing that the bearing was probably not influenced by local attraction. Let side 2 be the one so selected; assume its bearing N 75° 32′ E, as taken on the ground, to be correct; through either end of it, say at its farther end 2, draw a short meridian line, parallel to which draw others through every corner.

Now, having the bearing of side 2, N 75° 32′ E, and requiring that of side 3, it is plain that the reverse bearing from corner 2 is S 75° 32′ W, and that therefore the angle 1 2 *m* is 75° 32′. Therefore, if we take 75° 32′ from the entire corrected angle 1 2 3, or 144° 57′, the remainder 69° 25′ will be the angle *m* 2 3; consequently the bearing of side 3 must be S 69° 25′ E. For finding bearing of side 3, also equal to 69° 25′, and if we add this to the entire corrected angle 2 3 4, or to 69° 32′ we have the angle a 3 4 = 69° 25′ + 69° 32′ = 138° 57′, which, taken from 180°, leaves the angle b 3 4 = 41° 3′; consequently the bearing of side 4 must be S 41° 3′ W.

For the bearing of side 5, we now have the angle 3 4 c = 41° 3′, which, taken from the corrected angle 3 4 5, or 120° 43′, leaves the angle c 4 5 = 79° 40′, consequently the bearing of side 5 must be N 79° 40′ W. At corner 5, for the bearing of side 6, we have the angle 4 5 d = 79° 40′, which, taken from 133° 10′, leaves the angle d 5 6 = 53° 30′; consequently the bearing of side 6 must be S 53° 30′ W; and so with each of the sides. Nothing but careful observation is necessary to see how the several angles are to be employed at each corner.

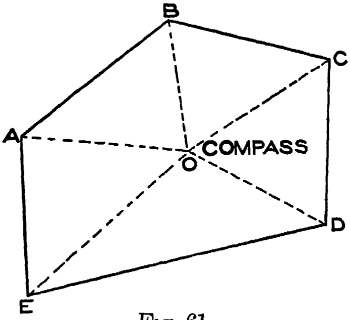


Fig. 61.

FARM SURVEYING.

Method of Progression. Farm surveying with the compass does not differ in any essential particular from the methods outlined for surveying a series of lines. If the boundary lines are irregular, it will be necessary to measure offsets at proper intervals, that the included area may be calculated. The method above described is known as the method of progression.

Method of Radiation. The method of radiation consists in setting up the instrument at some point inside or outside the field, from which all the corners are visible and accessible, and then measuring the bearing and lengths of the lines to these corners. Fig. 61 illustrates the method. Set up the compass at the point O, and take the bearings and lengths of the lines O A, O B, O C, O D, and O E.

Method of Intersections. Lay off a base-line of convenient length inside or outside the field, from which all the corners are visible. Set up the compass at one end of the base-line, and take the other end of the base-line, and take the bearings each corner in succession. Take the bearing and length ase-line. Now, when these bearings and lengths are the intersections of the lines will define the corners.

oofs of Accuracy. When the survey of a field is plotted, if the end of the last course meets the starting point, it proves the work, and the survey is then said to "close." Errors of closure may be due either to incorrect lengths of lines or to incorrect bearings, or to both.

Diagonal lines running from corner to corner of a field may be measured and their bearings taken. When these are plotted, their meeting the points to which they were measured proves the accuracy of the work.

Finally, the accuracy of the work may be tested by calculating the "latitudes and departures" of all the courses. If their algebraic sum is equal to zero, the work is correct. A check upon the bearings may be had by calculating the "deflection angles" between the courses. If their sum is equal to 360 degrees, the bearings are correct. This, however, will seldom be the case. A certain amount of error is permissible, depending upon the nature and importance of the work.

Field Notes. The field notes may be recorded in various ways, the object being to make them clear and full.

1. The surveyor may make, in the field book, a rough sketch of the survey by eye, and note on the lines their bearings and lengths. If a protractor and scale are available, the actual bearings and lengths of the lines may be plotted in the notebooks, as well as offsets, etc.

2. Draw a straight line *up* the page of the notebook, and record on it the bearings and lengths of the lines. Offsets, tie-lines, etc., can be plotted in their proper positions.

3. Write the stations, bearings, and distances in three columns. This method has the advantage, when applied to farm surveying, of being convenient for use in the subsequent calculation of contents, but does not give facilities for noting effects. It is illustrated as follows:

STATIONS.	BEARINGS.	DISTANCES.
0	N. 32° E.	16.82
1	S. 36° E.	18.90
3	S. 27½° W.	7.85
4	S. 16° W.	15.30

Notice that distances are given in Gunter's chains, and in calculating content the result will be given in square chains, which can be reduced to acres by pointing off one decimal place.

To Change Bearings. In certain kinds of work with the compass, it is convenient to assume one of the lines as a meridian, and it then becomes necessary to change the bearings of all of the other lines to conform with the assumed meridian. This case is best illustrated by an example.

The bearings of the sides of a field are here shown: Suppose now that the first course is assumed as a meridian, that is, that its bearing is due north and south. Required the bearings of the remaining courses.

STATIONS.	BEARINGS.	DISTANCES.
1	N. 35° E.	2.70
2	N. 83½° E.	1.29
3	S. 57° E.	2.22
4	S. 34¼° W.	3 55
5	N. 56½° W.	3.23

Since the courses are changed to the west by 35°, the new bearing of course 2 will be N 48½° E. Of course 3 it will be 57° + 35° = 92°, or the new bearing will be N 88° E. Of course 4 it will be 34¼° — 35°, or ¾° in the next quadrant, or the bearing will be S ¾° E. Of course 5 it will be 56½° + 35° = 91½°, or the bearing will be S 88½° W.

EXAMPLE FOR PRACTICE.

The bearings of a series of courses are given as follows:

It is required to determine the bearings of all the courses, due to this change. Find bearings and plot the lines.

Ans. Course 2 = N 62¼° E; 3 = N 9° W; 4 = N 68° W.

STATIONS.	BEARINGS.	DISTANCES.
1	S. 21° W.	12.41
2	N. 83¼° E.	5.86
3	N. 12° E.	8.25
4	N. 47° W.	4.24

Latitudes and Departures. The **latitude** of a point is its distance north or south of some line taken as a *parallel of latitude*, or line running east and west.

The **longitude** of a point is its distance east or west of some line taken as a *meridian*, or line running north and south.

The distance that one end of a line is north or south of the other end is the "Difference of Latitude" of the two ends of the line, and is called its **northing** or **southing**, or its **latitude.**

The distance that one end of a line is east or west of the other end is the "Difference of Longitude" of the two ends of the line, and is called its **easting** or **westing**, or its **departure.**

The terms **Latitude Difference** and **Longitude Difference** have of late come into quite general favor; but while they are perhaps more explicit, they are certainly cumbersome, and the older terms will be adhered to in what follows.

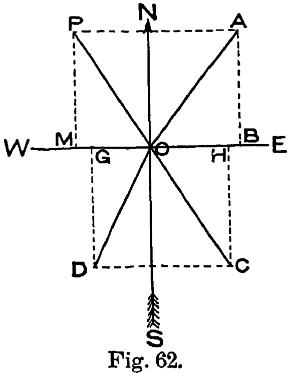


Fig. 62.

In Fig. 62, N S represents a meridian, and E W a parallel of latitude. If we take the line O A, its bearing as given by the compass is the angle NOA. The latitude or northing of the point A is therefore A B = O A cos NOA. Its departure or easting is O B = O A sin NOA.

To find the *latitude* of a course, multiply the length of the course by the natural *cosine* of the bearing; and to find the *departure* of any course, multiply the

If the course be northerly, the latitude will be north, and will be designated by the sign +, or plus; if the course be southerly, the latitude will be south, and will be designated by the sign —, or minus.

If the course be easterly, the departure will be east, and will be designated by the sign +, or plus; if the course be westerly, the departure will be west, and will be designated by the sign —, minus

Thus in the figure, OA is of plus latitude and plus departure; OP is of plus latitude and minus departure; OD is of minus latitude and minus departure; and OC is of minus latitude and plus departure.

For calculating latitudes and departures, a set of traverse tables may be procured; but a table of natural functions will be satisfactory, though possibly less convenient.

Testing a Survey by Latitudes and Departures. It is evident that after the surveyor has gone completely round a field or farm, measuring all the lengths and bearings, returning to the starting point, he has gone as far north as south, and as far east as west. In other words, if the work has been done correctly, the algebraic sum of the latitudes must equal zero, and the algebraic sum of the departures must equal zero. This condition, however, will seldom be attained, and it becomes necessary to decide how much error may be permitted without necessitating another survey. This will depend upon the nature of the work and its importance, and a surveyor will soon determine for himself his factor of error, depending partly upon his instrument, partly upon personal skill, for ordinary cases. If it is necessary to depend upon a "green" hand to carry the tape or chain, this may prove a fruitful source of error.

We shall now proceed to calculate the latitudes and departures of the survey as given below. Arrange the diagram as below with seven columns:

STATIONS.	BEARINGS	DISTANCES.	LATITUDES.		DEPARTURES.	
			N.	S.	E.	W.
1	S. 21° W.	12.41	...	11.591	.. .	4 443
2	N. 83¼° E.	5.86	0.691	......	5 819	.. .
3	N. 12° E.	8.25	8 069	. ....	1 716	. ..
4	N. 47° W.	4.24	2 892	. ....	. . .	3.104
		30 76	11 652	11 591	7 535	7.547

The cosine of the bearing of course 1 is 0.934.12.41=11.591 —Latitude.
The sine of the bearing of course 1 is 0.358.12.41= 4.443—Departure.
The cosine of the bearing of course 2 is 0.118.5.86 = 0.691 + Latitude.
The sine of the bearing of course 2 is 0.993.5.86 = 5.819 + Departure.
The cosine of the bearing of course 3 is 0.978.8.25 = 8.069 + Latitude.
The sine of the bearing of course 3 is 0.208.8.25 = 1.716 + Departure.
The cosine of the bearing of course 4 is 0.682.4.24 = 2.892 + Latitude.
The sine of the bearing of course 4 is 0.732.4.24 = 3.104—Departure.

The latitudes fail to balance by 0.061 chains, and the departures by 0.012 chains. The error of "closure" of the survey is therefore $E = \sqrt{.061^2 + .012^2} = 0.062 +$ chains, or approximately 4.09 feet. This sum may be divided up among the courses in proportion to the length, or the bearings may be corrected, or partly one and partly the other, as will hereafter be explained.

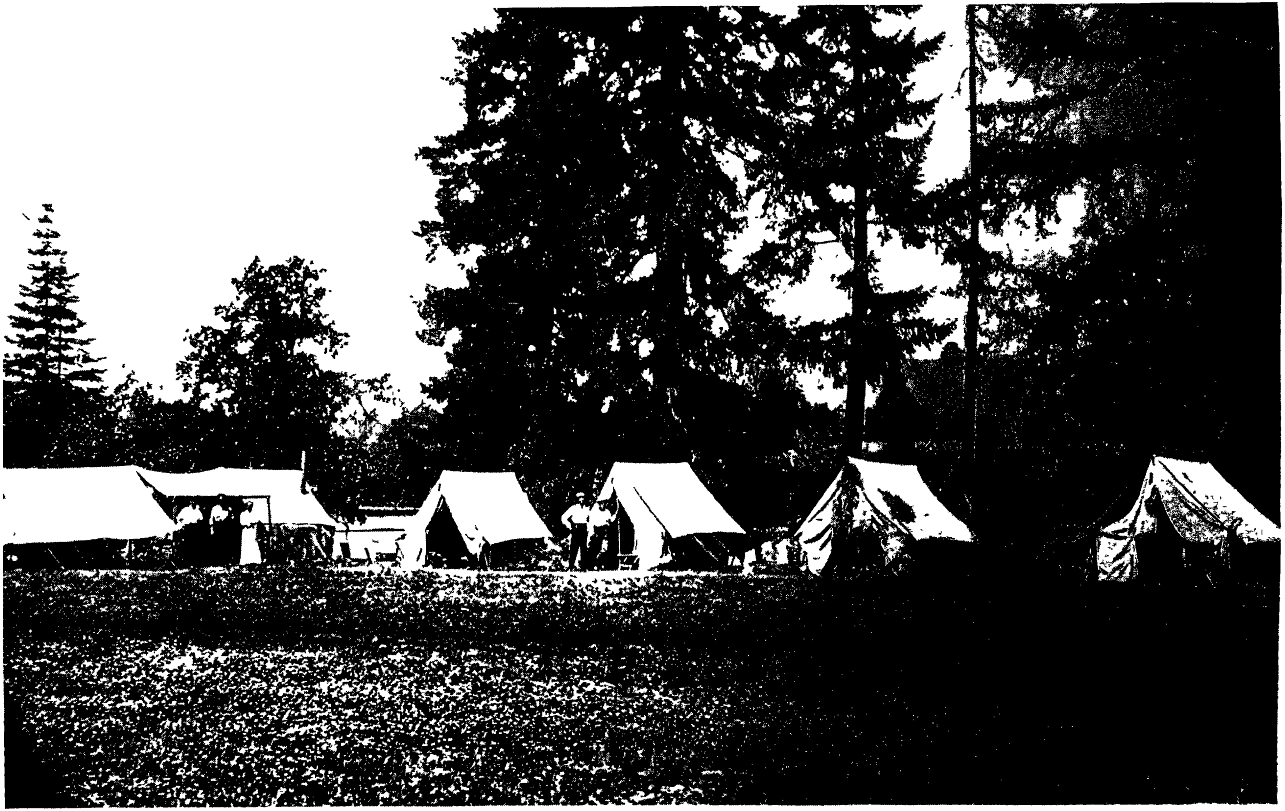
Balancing the Survey. Before proceeding to the calculation of the content of a field or farm, the survey must be balanced; that is, the latitudes and departures must be corrected so that their sums shall be equal, or shall balance. As to whether the bearings or lengths shall be corrected, will depend somewhat upon the conditions under which the survey was made. If the surveyor has reason to think that the error is entirely in the bearing of one or more, or even of all of the courses, the corrections may be made accordingly. If, on the other hand, one or more of the courses were measured over difficult ground, it may be presumed that the error occurred in those lines. If, however, there is no reason to believe that one course is in error more than another, the differences may be distributed among the courses in proportion to their length, according to the following proportions:

As the length of any course is to the sum of the lengths of all the courses, so is the correction of the latitude of that course to the total error in latitude of all the courses.

As the length of any course is to the sum of the lengths of all the courses, so is the correction of the departure of that course to the total error in departure of all the courses.

The practical application of these proportions to balancing a survey will be illustrated from the preceding problem:

For course 1... 12.41 : 30.76 :: x : 0.061... $x = .0246$, correction for latitude.
For course 2.... 5.86 : 30.76 :: x : 0.061....$x = .0116$, correction for latitude.



TOPOGRAPHIC ENGINEERS' CAMP AT WILLAMETTE VALLEY, OREGON

Courtesy of United States Geological Survey, Washington, D. C.

For course 3.... 8.25 : 30.76 :: x : 0.061....x = .0164, correction for latitude.
For course 4.... 4.24 : 30.76 :: x : 0.061....x = .0084, correction for latitude.

Since the sum of the north latitudes is the greater, the corrections will be subracted from them and added to the south latitudes. That is to say, the correction for course 1 will be added to 11.591, the result being 11.6156. The correction for course 2 will be subtracted from 0.691; that for course 3 will be subtracted from 8.069; and so on.

For course 1...12.41 : 30.76 :: x : 0.012...x = .0048, correction for departure.
For course 2... 5.86 : 30.76 :: x : 0.012...x = .0023, correction for departure.
For course 3.... 8.25 : 30.76 :: x : 0.012...x = .0032, correction for departure.
For course 4.... 4.24 : 30.76 :: x : 0.012...x = .0017, correction for departure.

The corrections are to be subtracted in this case from the west departure and added to the east departure.

In this example, the errors are small, but often they will be so large as to raise doubt as to the accuracy of the survey. In such a case, go carefully over all the computations, and, if the error is still too large, check the exterior angles of the figure (their sums should equal 360°), and if necessary repeat the survey. Having corrected the latitudes and departures, the corrected bearings of the courses may be deduced from the trigonometric ratio:

$$\text{Tan. bearing} = \frac{\text{corrected departure}}{\text{corrected latitude}}.$$

Calculating the Content. After a field has been surveyed, its content may be calculated by dividing it up into triangles, trapezoids, etc., calculating the various contents, and adding them together. This, however, is at best a cumbersome method, involving much work of calculation and great chance of error. The method of latitudes and departures is at once simple, easily applied, and easily checked.

Before proceeding to develop a formula for this method, it will be necessary to illustrate and define certain terms.

Draw a line, as N S (Fig. 63), through the extreme east or west corner of the field for a meridian. From the definitions previously given, the difference of longitude of the two ends of a line is the *departure* of the line. I B is therefore the departure of the line A B. The departure of the line B C is L C; that of E F is S F; and that of A F is O Q.

The perpendicular distance of each station from the given west. Thus the longitude of A is zero; that of B is I B; that of C is I B + L C; that of E is O Q + F S; and that of F is O Q = ZS — FS.

The difference of latitude of the two ends of a line is called the *latitude* of the line. Thus the latitude of A B is A I; that of B C is B L; that of E F is E S.

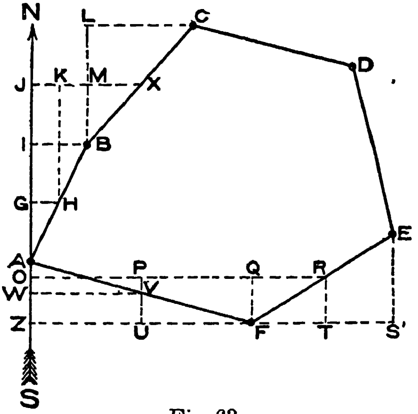


Fig. 63.

The distance of the middle of any side of a field from the meridian is called the *longitude of that side.* Thus the longitude of the side A B is G H; that of B C is J X = G H + K M + MX; and that of A F is WV = OR — QR — QP, the minus signs being used in this instance because the lines E F and A F bear to the west. An analysis of W V will show that it equals O R (longitude of preceding course) + [— R Q (one-half departure of preceding course)] + [— QP (one-half departure of the course itself)].

To avoid fractional quantities, double the preceding expressions and then deduce a general rule for finding **double longitudes.**

The double longitude of the first course equals the departure of that course. The double longitude of the second course equals the double longitude of the first course, plus the departure of the first course, plus the departure of the second course.

The double longitude of any course equals the double longitude of the preceding course, plus the departure of the preceding course, plus the departure of the course itself.

We shall now proceed to deduce a rule for determining areas by double longitudes and departures, and shall first take a three-sided field, as in Fig. 64.

Drawing a line through the most westerly corner A, we see that the area of the field will be the difference between the area of the trapezoid D B C M and the combined area of the triangles product of D B by D A, or the double longitude of A B by the latitude of A B. The resulting product will be north or *plus*. The double area of the trapezoid D B C M is the product of (D B + M C) = 2 G H, by D M, that is, the double longitude of B C by its latitude. The resulting product will be south or *minus*. The double area of the triangle A C M will be the product of M C by A M, or the double longitude of the course A C by its latitude. The resulting product will be north or *plus*. Adding together, then, the plus products, and subtracting from the minus product, gives as the result the double area of the field.

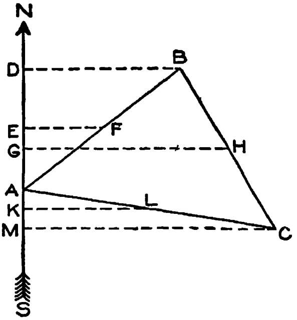


Fig. 64.

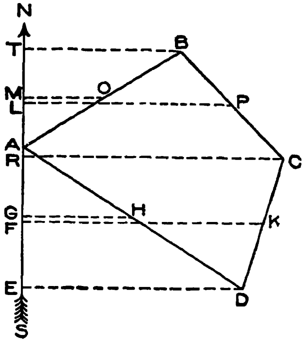


Fig. 65.

We shall next take a four-sided field, as in Fig. 65.

It is evident that the area of the field A B C D is the difference between the sum of the areas of the two trapezoids T B C R and R C D E and the sum of the areas of the triangles A B T and A D E.

The double area of the triangle A B T is the product of B T by A T, or the double longitude of the course A B by its latitude. The result will be a north product or *plus*. The double area of the trapezoid T B C R will be the product of (T B + C R) = 2 L P by T R—that is, the double longitude of the course B C by its latitude. The result will be a south product or *minus*. The double area of the trapezoid R C D E will be the product of (R C + D E) = 2 F K, by R E, or, the double longitude of the course C D

STATIONS	BEARINGS	DISTANCES	LATITUDES		DEPARTURES		DOUBLE LONGITUDES	DOUBLE AREAS	
			N.	S.	E.	W.		+	—
1	S 21° W.	12 41		11 616		4 438	—7.204	83.682	
2	N. 83¼° E.	5 86	0 679		5.821		—5.821		3.952
3	N. 12° E.	8 25	8.053		1 719		+1.719	13 843	
4	N. 47° W.	4 24	2 884			3 102	+0 336	0 969	
							98 494		3.952
							3 952		
						2	94 512		
							47 271		

AREA=47.271 SQ. CHS.=4 ACRES, 2 ROODS, 37 SQ. PERCHES.

double area of the triangle A D E will be the product of E D by A E, or the double longitude of the course A D by its latitude. The result will be a north product or *plus*. Finally, adding together the north products, adding together the south products, and taking the difference of their sums, gives as the result the double area of the field A B C D.

The same principle will apply to any enclosed area, however great the number of the sides. *The area will always be one-half the difference of the sums of the north and the south products arising from multiplying the double longitude of each course by its latitude.*

For systematic computation arrange the work as follows:

Arrange the columns as in the problem on page 83.

Balance the latitudes and departures, putting the corrected quantities above the others in red ink; or else arrange four additional columns, and enter them in their proper places.

Compute the double longitude of each course with reference to a meridian passing through the extreme east or west station. and place the results in another column.

Multiply the double longitude of each course by the corrected latitude of that course, and place north products in one column and south products in another.

Add together the north products and also the south products, and take the difference of their sums. Divide the difference by two, and the result will be the area desired.

If the survey has been made with a Gunter's chain, the result will be in square chains. Divide by ten to reduce to acres.

To test the correctness of the calculation, assume the meridi through the extreme station upon the other side of the field, an the work in detail as before.

We shall now proceed to calculate the content of the field g by the notes on page 81. The corrections to the latitudes wil found on page 82, and the corrected departures on page 83.

The arrangement of the columns for convenient calculation is as described on page 86. Upon making a rough sketch of the courses, it is found that station 3 is the farthest east; and therefore the double longitudes will be calculated beginning with course 3. From the definition previously given, the double longitude of course 3 is equal to its departure = + 1.719. The double longitude of course 4 equals the double longitude of course 3, plus the departure of course 3, plus the departure of course 4 = 1.719 + 1.719 + (− 3.102) = + 0.336. The double longitude of course 1 equals the double longitude of course 4, plus the departure of course 4, plus the departure of course 1 = 0.336 + (− 3.102) + (− 4.438) = − 7.204. The double longitude of course 2 equals the double longitude of course 1, plus the departure of course 1, plus the departure of course 2 = + (− 7.204) + (− 4.438) + 5.821 = − 5.821. Multiplying these double longitudes by their respective latitudes, gives the quantities in the last two columns, the first, third, and fourth being positive, and the second negative. Taking the difference of the sums of the quantities in these columns, and dividing the result by 2, gives the content of the field, 47.271 square chains. Dividing by 10 gives 4.7271 acres. Reduce to roods and perches by multiplying the decimal part by 4 and 40 successively.

The result may now be checked by beginning with the most westerly station, and it will be necessary to recalculate the quantities in the last three columns.

The following problems are taken from "Gillespie's Surveying" (Staley):

EXAMPLES FOR PRACTICE.

Calculate the content of the fields from the data tabulated below. The result, where found in square metres, should be reduced

(1)

STATIONS.	BEARINGS.	DISTANCES.
1	N. 34¼° E.	2.73
2	N. 85° E.	1.28
3	S. 56¾° E.	2.20
4	S. 34¼° W.	3.53
5	N. 56½° W.	3.20

Ans. 1 acre, 3 roods, 19 perches.

(2)

STATIONS.	BEARINGS.	DISTANCES.
1	N. 35° E.	2.70
2	N. 83½° E.	1 29
3	S. 57° E.	2.22
4	S. 34¼° W.	3.55
5	N. 56½° W.	3.23

Ans. 1 acre, 0 roods, 15 perches.

(3)

STATIONS.	BEARINGS.	DISTANCES.
1	S. 5° 35′ W.	2,388 88 meters
2	S. 39° 35′ W.	1,060 27 meters
3	S. 50° 25′ E.	3,078 31 meters
4	S. 79° 5′ E.	325 00 meters
5	S. 53° 50′ E.	275 00 meters
6	S. 48° 15′ W.	200 00 meters
7	N. 82° 45′ E.	450 00 meters
8	S. 87° 40′ E.	186.72 meters
9	N.	8,768 12 meters
10	N. 84° 25′ W.	1,898 54 meters
11	S. 5° 35′ W.	3,530.60 meters
12	N. 84° 25′ W.	257 50 meters

Ans. { 4,999 acres
3 roods.
$39\frac{96}{100}$ sq. rods.

Supplying Omissions. The method of latitudes and departures may be applied to supplying any two omissions in the field notes, as will be explained in connection with the "Use of the Transit."

Azimuth. The azimuth of a line is the horizontal angle which the line makes with some other line taken as a meridian. It differs from bearing in that it is measured continuously from 0° to 360°. All descriptions of property must be given in terms of bearings, but line surveys with either the compass or the transit had better be given in terms of the azimuth.

In astronomical and geodetic work it is customary to reckon azimuth from the *south point around through the west*, through 360°. For the ordinary operations of surveying, however, it is better to measure the azimuth from the north point to the right through 360°.

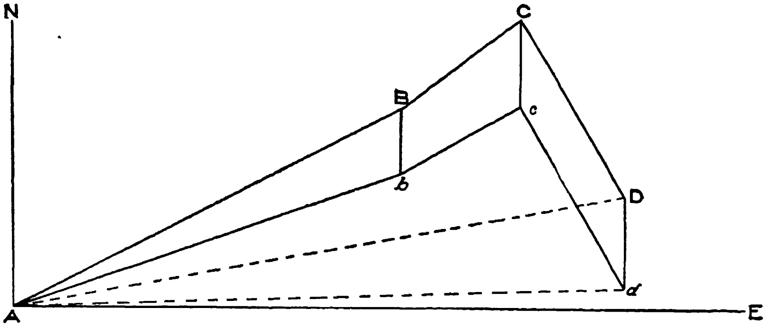


Fig. 66.

RESURVEYS.

Where the boundary lines of a farm or town have been obliterated and the corners lost, it is often necessary to make resurveys in order to re-establish them. If the corners can be found by reliable evidence, they must be accepted as corners even though the second bearings and lengths of the lines indicate different points.

It sometimes happens that some corners can be found while others cannot. In such cases a series of random lines is to be run with the old bearings, or with the old bearings corrected for a change in declination of the needle between the two dates.

As an example, let the records in an old deed give the length

Ab N 60° E 10 chains.
bc N 45° E 4 chains.
cd S 45° E 8 chains.

There being no definite data at hand to determine the change in the magnetic declination between the dates of the two surveys, the lines AB, BC and CD are run with the given bearings and distances from the known corner A. The old corners *b* and *c* cannot be found; but on arriving at D the old corner *d* is discovered at a point 20.4 links S and 12° W from D It is required to locate the old corners *b* and *c*.

By the method of latitudes and departures explained before, the lengths of the lines DA and *d*A may be computed. They are: for DA, south 82° 47′, west 17.29 chains; for *d*A, south 83° 26′, west 17.22 chains.

Now the error D*d* between the two corners is due to two causes: (1) the continued variation in the magnetic bearings of the old surveys, (2) the difference in the length of the chains used. The first cause alters the polygon A*bcd*A around the point A by a small angle. The second cause alters the length of the sides in a constant ratio. The difference between the bearings DA and *d*A is the constant angle, while the ratio of the length of the old lines is the constant ratio. To find the bearings of the old line, therefore, each of the given bearings is to be corrected by the amount 83° 26′ minus 82° 47′ = 0° 39′. To find the length of the old line each of the given lengths is to be multiplied by $\frac{17.22}{17.29} = 0.996$.

Suppose now that the work of computation has been done with such precision that the error in chaining must be regarded as lying in the old survey. Applying these results, we find the adjusted bearings and lengths of the old line to be,

Ab = N 60° 39′ E 9.96 chains.
bc = N 45° 39′ E 3.99 chains.
cd = S 44° 21′ E 7.97 chains.

With the new data the line may be rerun and the corners *b* and *c* located, a check on the field work being that the lost line should end exactly at *d*.

It is, however, not difficult to compute the length and bearings

Since the angle *B A b* is small, the triangle *B A b* may be considered similar to the triangle *D A d*. We will then have the proportion:

$$B\,b : D\,d :: A\,B : A\,D.$$

Or,

$$B\,b = D\,d \times \frac{A\,B}{A\,D} = \frac{20.4 \times 10}{17.29} = 11.8 \text{ links.}$$

A similar proportion may be written for the side *B c*, and the result added to the value of *B b*.

The same principle may be used to determine the bearings of *B b* and *C c*, so that, with the lengths and bearings of these lines determined, the most probable location of the old corners *b* and *c* can be fixed.

EXAMPLE FOR PRACTICE.

The records of an old survey read as follows:

"Commencing at a point marked No. 5 and running N 62° E 14 chains to a stake marked A, thence running N 43½° E 8.00 chains to a stake marked B, thence N. 5° W. 12.00 chains to a stake C, thence N 72½° E. 10.25 chains to a stake D, thence S 12° W 6.43 chains to a stake marked No. 3. On running the lines, the end of the last one, instead of being at a stone marked No. 3, was 0.62 chain due E from it." Find the adjusted bearings and lengths of the old lines; also find the distance and direction from each station of the new survey to the corresponding corner of the old.

DIVISION OF LAND.

The method of latitudes and departures is especially useful in the division of land. The problem is usually as follows: Given the lengths and bearings of the sides of a field containing a certain area; it is required to divide the area into certain parts by a line running in a certain direction, in which case it is necessary to determine the starting point of the dividing line. Or it is required that the line shall begin at a certain point, in which case it is necessary to determine the direction of the line.

A certain field is described as follows:

1	N. 63° 51′ W.	6.91 Chs.
2	N. 63° 44′ W	7.26 "

4	N. 77° 50′ W.	6.54 Chs.
5	N. 31° 24′ E.	14.38 "
6	N. 31° 18′ E.	16.81 "
7	S. 68° 55′ E.	13.64 "
8	S. 68° 42′ E.	11.54 "
9	S. 33° 45′ W.	31.55 "

Beginning at a point upon the side 91, it is required to divide the area into two equal parts of 37 acres by a line having a bearing of N. 68° 46′ W. Some preliminary calculations will be necessary to determine the starting point of the dividing line such that the resulting area will be nearly that desired, and this dividing line will be practically parallel to a line connecting stations 7 and 9. Call the dividing line *A B* (see Fig. 67), and if this line begins at a point 15 chains from station 9, the length of *A B* will be 24.528 chains; that of *B* 7 will be 14.90 chains. We shall now proceed to calculate the area by latitudes and departures, beginning with station 9. The calculations are shown in the following table, and the result, 36.5186 acres, is short of the required area by 0 4814 acre or 4.814 square chains.

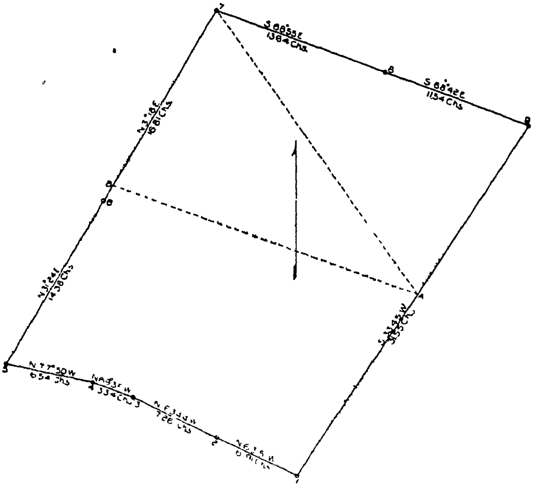


Fig. 67.

STATION	BEARINGS	DISTANCES	LATITUDES N	LATITUDES S	DEPARTURES E	DEPARTURES W	DOUBLE LONGITUDES	DOUBLE AREAS +	DOUBLE AREAS —
9	S 33° 45′ W	15 00		12 472		8 334	— 8 334	103 912	
A	N 68° 46′ W	24 528	8 883			22 863	- 39 531		351 154
B	N 31° 18′ E	14 90	12 731		7 741		—54 653		695 787
7	S 68° 55′ E.	13 64		4 907	12 727		—34 185	167 746	
8	S 68° 42′ E	11 54		4 192	10 752		—10 706	44 880	
								316 568	1046 941
									316 568
								2	730 373

Our line *A B* must therefore be moved farther south such a distance that the area enclosed between the first and second positions shall equal 4.814 square chains. Considering this area as a rectangle (which it will be nearly), the line *A B* will be moved south $\frac{4.814}{24.528}$ = 0.1961 chain, or, say, 0.2 chain. The line 9 *A* will be therefore 15.2 chains in length. The length of *A B* will not be materially changed. *B* 7 will now be 15.1 chains in length, while 78 and 89 will be the same as before. The latitudes and departures of 9 *A*, *A B*, and *B* 7 must be recalculated, as well as the double longitudes of all of the courses. The calculations are found in the following table, and there results an area equal to 37.015 acres.

STATION	BEARINGS	DIS-TANCES	LATITUDES		DEPARTURES		DOUBLE LONGI-TUDES	DOUBLE AREAS	
			N.	S.	E.	W.		+	—
9	S 33° 45′ W.	15 20		12 638		8 445	— 8 445	106.728	
A	N. 68° 46′ W	24.528	8 883			22.863	—39.753		353 126
B	N 31° 48′ E	15 10	12 902		7 845		– 54 771		706 655
7	S 68° 55′ E	13 64		4.907	12 727		—34 199	167.814	
8	S 68° 42′ E	11 54		4.192	10 752		– 10.720	44 938	
								319 480	1059 781
									319.480
								2	740 301
									370 150

AREA = 37.015 ACRES.

Referring to Fig. 67, starting at a point *A* on 91, 12.25 chains from station 9, it is required to find the length and bearing of *A B* such that the area 9 *A B* 7 8 9 shall equal 32 acres.

First draw a line from *A* to station 7, and by latitudes and departures calculate the area *A* 7 8 9 *A*, and determine the length and bearing of *A* 7. Call this latter area *H*. Then the area *A B* 7 *A* must equal 32-*H*. From the point *B*, erect a perpendicular *B C* to the line *A* 7 The area of *A B* 7 *A* will equal $\frac{A\ 7}{2} \times B\,C$ = (32–

H); $\therefore BC = \dfrac{2\,(32\text{–}H)}{A7}$. Therefore $B7 = \dfrac{BC}{\sin B7C}$. In the triangle $A\,B\,7\,A$ we now have two sides and the included angle from which to calculate the length and bearing of $A\,B$.

THE TRUE MERIDIAN.

In order to ascertain the true meridian of a given place, several methods may be pursued. The general practice is to use the star Polaris at culmination or elongation. This star is on the , nearly, when a plumb line covers it and the star Zeta, the next to the end of the handle of "The Dipper." See Fig. 68.

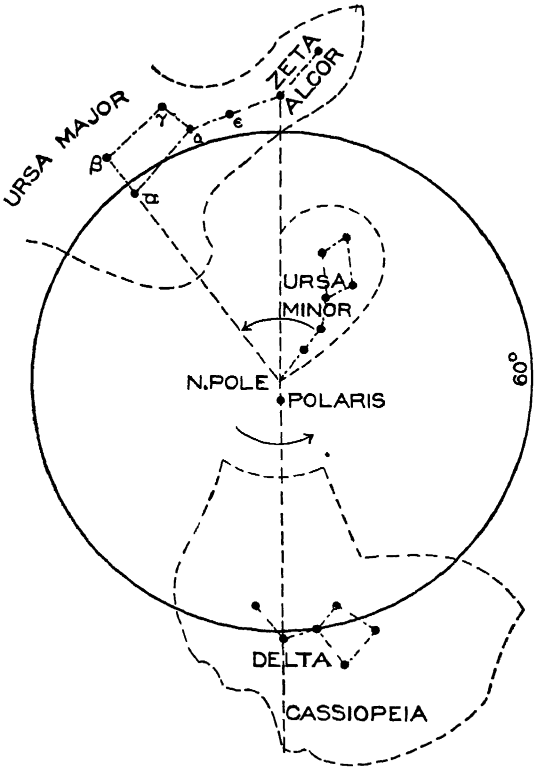


Fig. 68.

When Polaris is on the meridian, as illustrated in this instance, it is said to be at "culmination." This star is often referred to as the north or pole star. It is about 1¼° from the pole, and revolves around the pole once every 23 hours and 56 minutes. Thus it is apparent that it comes on the true meridian twice each day. The arrows in the figure indicate the direction of the rotation.

To Determine the True Meridian by the Compass. *With Polaris at Eastern or Western Elongation.* To determine the true meridian by means of the compass, take a plumb-line, and attach one end of the line to any suitable support situated as far above the ground as practicable, so as to have a clear field of view about 20 feet away. A board nailed on a telegraph pole, tree, or post at right angles, will suffice for this purpose. The plumb-bob may be of any suitable material, of about 5 lbs. in weight, as a brick, stone, iron ring, or coupling. It will serve the same purpose, with as accurate results, as the most highly polished or carefully manufactured plumb-bob. The plumb-line should be about 25 feet in length, depending upon the latitude of the place, since the altitude of the pole above the horizon at any place is equal to the latitude of that place.

Illuminate the plumb-line just below its support by means of a bull's-eye lantern, lamp, or candle, care being taken not to obliterate the line from the view of the observer. The best way is to screen the light, and throw the light on the plumb-line by means of a reflector.

Next unfasten one of the uprights of the compass, and place it on a horizontal rest at some convenient point south of the plumb-line, say 30 feet in an east or west direction, and in such a position that when viewed through the peep-sight, Polaris will appear about two feet below the support of the plumb-line. It is customary to determine this position by trial the night before the observation.

About 25 minutes before the time of elongation, as per table on page 130, bring the peep-sight into the same line of sight with the plumb-line and the star Polaris. Before reaching elongation, the star will move away from the plumb-line, to the east for eastern elongation, and to the west for western elongation. Hence, by moving the peep-sight in the proper direction—that is, east or remain stationary, thus indicating that it has reached its point of elongation. The peep-sight will now be secured in place by a clamp or weight, with its exact position marked on the rest. Now defer all further operations until the next day.

The next morning place a slender flag or ranging pole at a distance of 200 or 300 feet from the peep-sight, and exactly in line with the plumb-line. Next carefully measure this distance, and take from the table (page 130) the azimuth of Polaris, corresponding to the latitude of the station of observation; find the natural tangent of this azimuth, and multiply it by the measured distance from the peep-sight to the rod. The product will express the distance to be laid off from the rod, exactly at right angles to the direction already determined—that is, to the west for eastern elongation, and to the east for western elongation; and this point with the peep-sight, will define the direction of the meridian with sufficient accuracy for the needs of local surveyors.

The position of the pole star may be found by means of the two stars β and α in the bowl of the "The Dipper" (Fig. 68), which are called the "pointers" because of their pointing approximately to the pole star.

THE TRANSIT.

Construction. The transit is used for measuring horizontal and vertical angles directly, and for measuring bearings indirectly. It consists of a telescope mounted in standards attached to a divided horizontal plate, the telescope serving to define accurately the line of sight; while the horizontal plate, divided into degrees, minutes, and twenty or thirty seconds of arc, makes it possible to measure small horizontal angles. The instrument is provided with a three- or four-screw leveling base, by means of which it is attached to the tripod.

The telescope is similar in construction to that of the Wye level, but is shorter and of less magnifying power, a power of from 24 to 26 diameters being about the average for the ordinary transit. The eye-piece may be either inverting or erecting, but the former is to be preferred.

Since the principal function of the transit is to secure align-plane, and to that end is supported in the standards by a transverse axis, permitting the telescope to be "transited," that is. turned through a complete vertical circle.

For measuring horizontal angles the instrument is arranged with an upper and a lower motion, sometimes called the upper and the lower "limb." The lower limb is supported by the leveling base by means of a hollow conical axis; and into it is fitted, in turn, the conical axis of the upper limb. Each limb may be turned independently of the other, or they may be clamped together and to the leveling base. The lower limb carries the divided circle and the upper limb the vernier. For ordinary purposes the circle is divided to one-half degrees, and reads to single minutes by means of the vernier. It may also be divided so as to read to 20 or 30 seconds, and occasionally to 10 seconds. The divisions of the circle, however, should not be so crowded as to render the reading difficult, and the graduations should be properly adjusted to the magnifying power of the telescope.

The verniers may be set at right angles to, or parallel with the line of sight, or at 30° thereto. With the verniers parallel with the line of sight—that is to say, directly under the telescope—or making an angle of 30° with the line of sight, the observer can read the angles without moving from his position, thereby avoiding the risk of disturbing the instrument by walking around it. See Fig. 69.

For leveling the instrument, there are provided two level tubes set at right angles to each other. These are shown in the figure. One of them is attached to the upper plate, while the other may be attached either to the upper plate or to one of the standards. On account of lack of space these level tubes are quite short.

The four-screw leveling base may consist of two parallel plates connected to each other by a one-half ball and socket joint, or the upper plate may be replaced by a ribbed casting. The four leveling screws rest in cups upon the lower plate and extend through the upper plate or casting. The leveling base is attached to the instrument proper, and the whole is attached to the tripod by screwing to a casting firmly attached to the legs. The vertical axis is furnished with a hook, to which may ment. A shifting center is also provided, by means of which, after the instrument has been approximately centered over a stake, it may be accurately adjusted by loosening the leveling screws and shifting the instrument upon the lower leveling base. See Fig. 69.

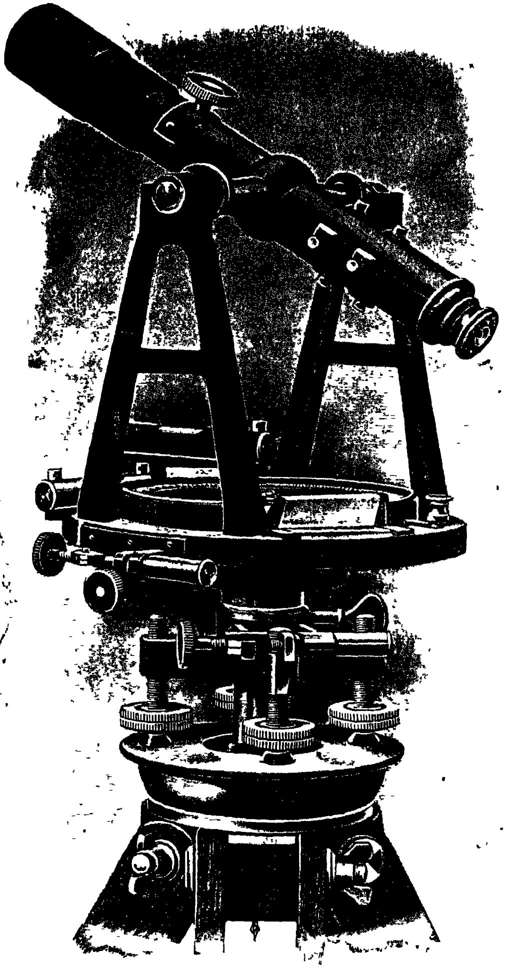
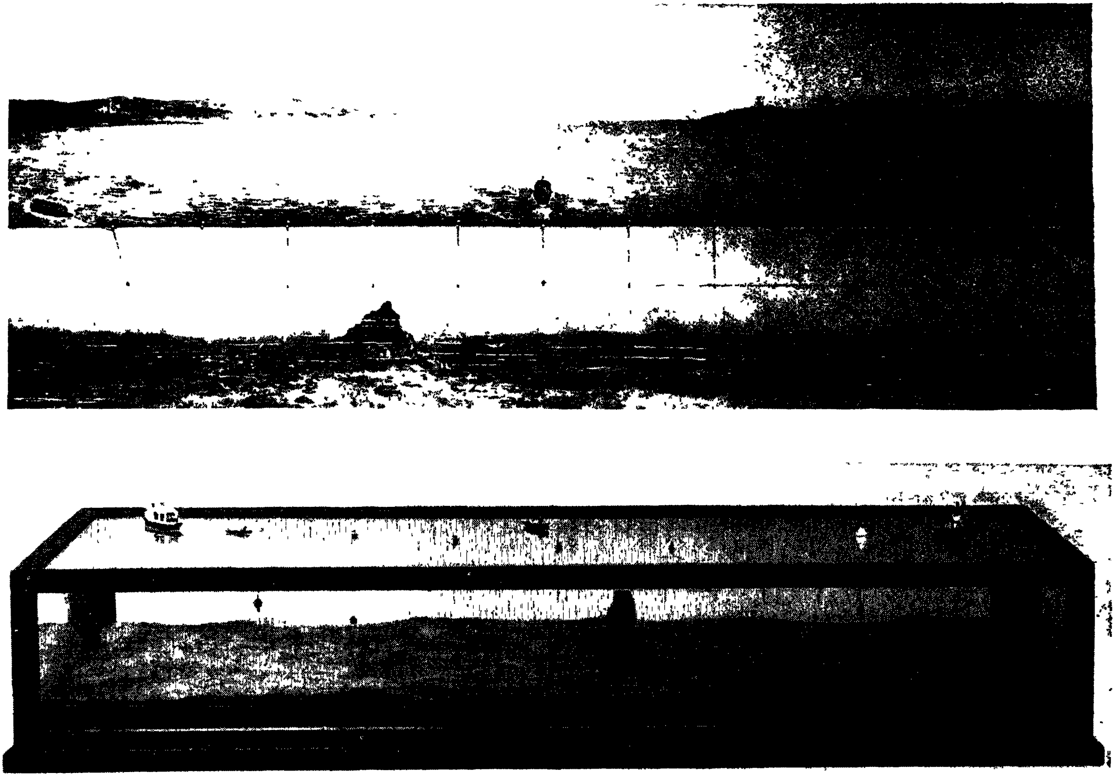


Fig. 69.

The three-screw leveling base is necessarily larger and differs in detail from the four-screw. The upper plate carrying the screws is permanently attached to the instrument; and the lower ends of



TWO VIEWS SHOWING METHOD OF OPERATING WIRE DRAG FOR THE DETECTION AND LOCATION OF ROCKS, SAND BARS, AND OTHER OBSTRUCTIONS

The upper view shows three boats attached to a line of buoys which carry the wire drag. The pile of rock is here shown to be at a safe depth In the model shown in the lower view the wire drag has caught on a rock, the obstruction being shown by the increased tension on the springs at each of the guiding boats, by the change in direction of the line of buoys from a perfect curve, and also by the tipping of the particular buoy nearest the obstruction The boat shown in the center and the two row boats leaving the larger vessels go to the obstructed point and make soundings, thereby locating the depth and nature of the obstruction.

Courtesy of United States Coast and Geodetic Survey, Washington, D. C.

the screws rest upon the tripod casting, to which it is attached by a single center screw fitted with a strong spiral spring that engages upon a thread cut upon the vertical axis of the instrument. See Fig. 72.

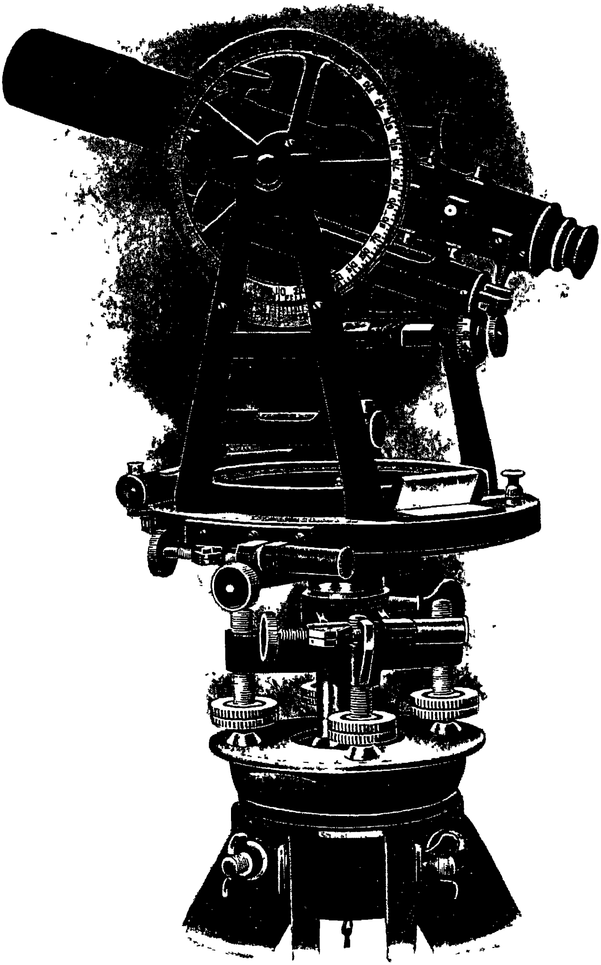


Fig. 70.

The four-screw base commends itself from the fact that it can quickly be leveled approximately; and, no matter how much the

The three-screw base, however, is more easily manipulated, and all danger of binding the screws and springing the plates is obviated. Whichever type of instrument is preferred, the screws should work

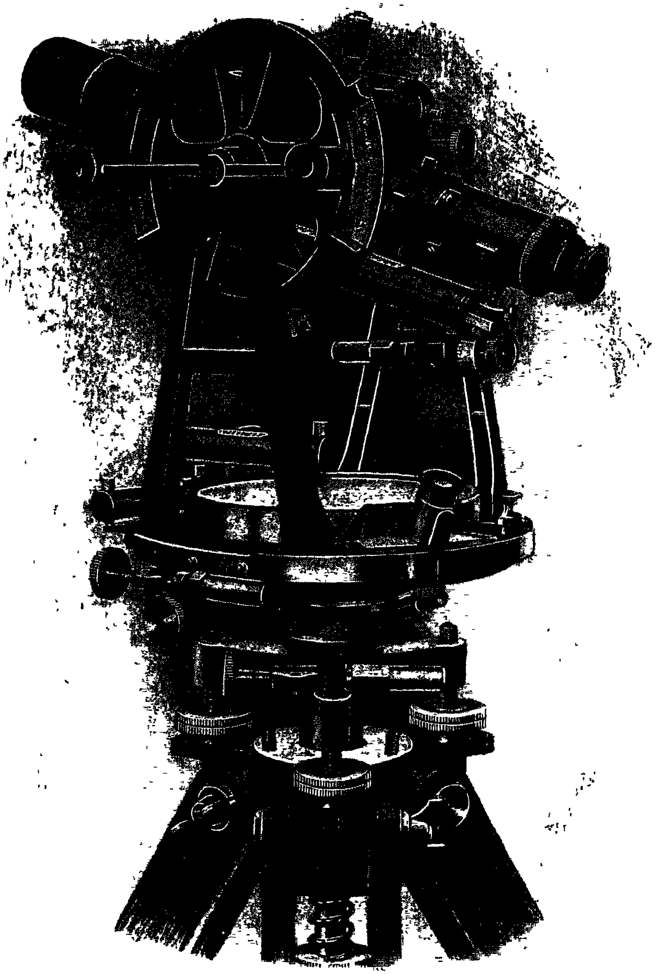


Fig. 71.

smoothly and evenly, and the pitch should be adjusted to the sensitiveness of the bubbles.

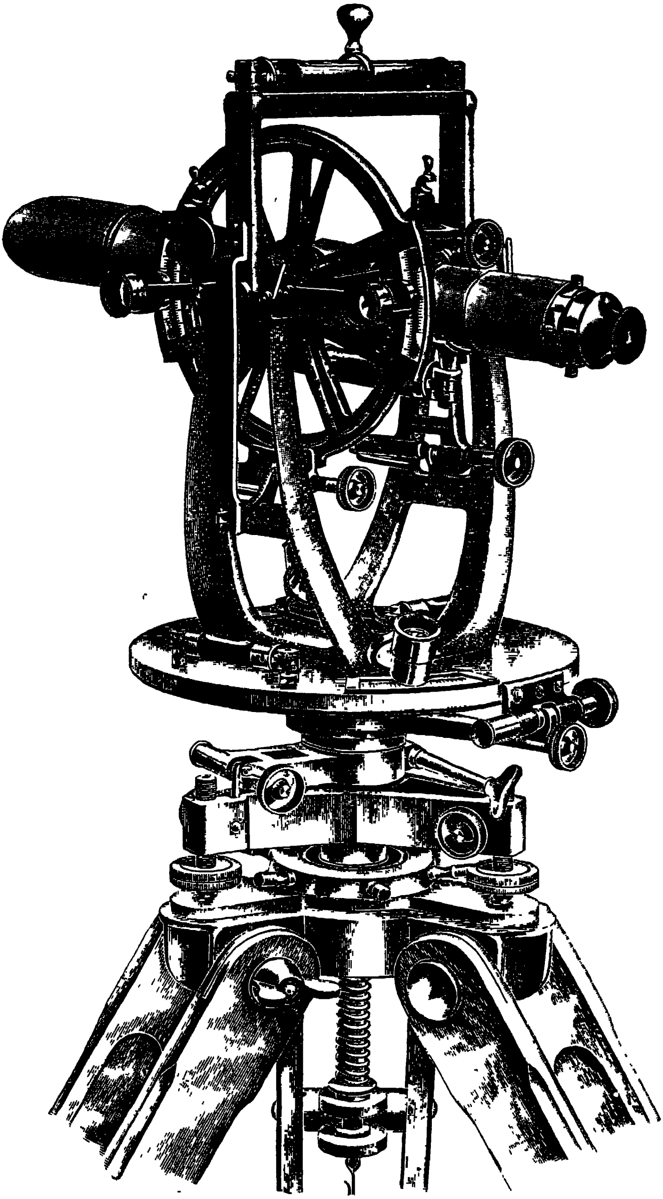


Fig. 72.

between the standards; but for city work, triangulation, etc., the

For measuring vertical angles, the transit is fitted with a vertical arc or circle divided usually to one-half degrees, and

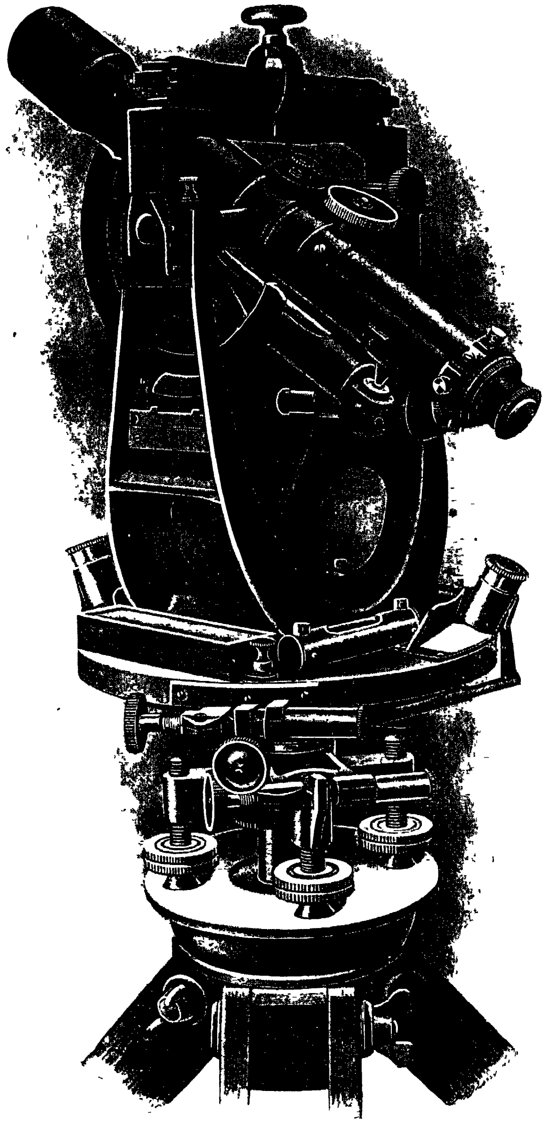


Fig. 73.

reading to single minutes by the vernier. A level tube may also be provided the transit may be used as a leveling instrument. A striding level resting upon the standards may also be provided, by means of which the instrument can be more accurately leveled than by the short levels upon the upper limb. See Fig. 70.

The telescope should always be provided with stadia wires, either fixed or adjustable, though the former are preferable. See article on "Stadia."

The gradienter screw is a device attached to the clamp of the telescope, by means of which grades can be established, and horizontal distances, vertical angles, and differences of level can be measured with great rapidity. See article on "Gradienter" in Part III.

Surveyor's Transit. This instrument is the plain transit, capable usually of measuring horizontal angles only, but occasionally fitted with a vertical circle or arc for measuring vertical angles. See Fig. 69.

Engineer's Transit. When the instrument is provided with a vertical circle or arc, a level underneath the telescope, with or without gradienter screw, it is called the engineer's transit. See Fig. 70.

Tachymeter. This term, meaning *rapid measurer*, has of recent years been applied to an instrument having a level attached to the telescope, a vertical arc or circle, and stadia wires. Such an instrument is adapted to the rapid location of points in a survey, since it is capable of measuring the three co-ordinates of a point in space, *i. e.*, the angular co-ordinates of altitude and azimuth, and the radius-vector or distance. The compass and gradienter are auxiliaries in the measurement of angles; and an instrument having them in addition to the essential features mentioned above, is more perfectly adapted for tachymetric work. See Fig. 71.

Theodolite. This term is applied to an instrument so constructed that the telescope will not transit, but, in order to take backward sights, the telescope must be lifted out of its supports and turned end for end. See Fig. 73.

Transit-Theodolite. This name is applied to an instrument in which the telescope not only can be transited, but also lifted as an angle-measurer it is desirable that the vertical cross-hair be at right angles to the horizontal axis of the telescope when the instrument is level.

To test this, set up the instrument at some convenient point, 200 or 300 feet from a wall, tree, or other convenient object, upon which a point is clearly defined by a tack or otherwise. Carefully level the instrument, and cover this point accurately with the lower extremity of the vertical hair. Clamp the horizontal axis of the telescope; and by means of the tangent-screw slowly move the telescope in a vertical plane, and note if the hair continues to cover the point from one extremity to the other. If it does, the hair is in its proper position. If not, loosen the diaphragm screws and turn the diaphragm vertically until the hair covers the point from end to end. This adjustment will disturb the last one and the two must be tested and corrected alternately until in perfect adjustment.

If the transit is to be used for leveling, it is necessary that the *horizontal* cross-hair be in the optical center of the object glass.

To test, set the instrument up firmly 200 or 300 feet from a wall, tree, or other convenient object, and, after leveling, carefully center the intersection of the cross-hairs upon a well-defined point. Clamp the axis of the telescope, turn the instrument upon its vertical axis through 180°, and carefully center a point upon the intersection of the cross-hairs in this new position. Clamp the vertical axis, unclamp the telescope axis, transit the telescope, and carefully center the intersection of the cross-hairs upon the first point. Now clamp the telescope, loosen the vertical axis, and again revolve the instrument through 180°. If the line of collimation again strikes the second point, the horizontal cross-hair is in adjustment. If not, carefully center this third point, bisect the distance between the second and third points, and move the cross-hair diaphragm until the intersection of the cross-hairs covers this fourth point. This adjustment will disturb the last two, and the three must be repeated in succession until accurately adjusted.

3d. *To make the horizontal axis of the telescope perpendicular to the vertical axis of the instrument.*

To test, set up the instrument as explained for the last adjustment, and, after leveling, center carefully a point at each extremity axis and transit the telescope, bringing one extremity of t horizontal cross-hair upon one of the points previously established. If the other extremity coincides with the second point, the axis of the telescope is in adjustment.

If the axis is out of adjustment, the method of procedure is best illustrated by Fig. 75.

A A′ is the line covered between the extremities of the horizontal wire or hair when the axis of the telescope is in adjustment. If it is not in adjustment, the wire will in the first position of the telescope cover the line A B, and in the second position the line A B′. Therefore bisect the distance B B′, and raise or lower the adjustable end of the telescope axis until the wire covers A A′. Now repeat the test, and the correction if necessary.

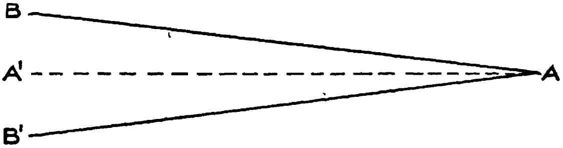


Fig. 75.

4th. *To make the axis of the telescope bubble tube parallel to the line of collimation of the telescope.*

This adjustment should be tested and corrected by the "peg" method as follows: Select a piece of comparatively level ground, drive a stake, "set up" the transit over it, and carefully level by the *plate* levels. Next drive two stakes into the ground, one in front of the transit and the other at the same distance behind it. In Fig. 76, C is the position of the transit, and A and B are two stakes, each 150 feet from C. D is a fourth stake behind B and in line with it from C. The transit being leveled by the plate levels, bring the bubble of the telescope tube to the center of the tube, by means of the tangent-screw attached to the horizontal axis of the

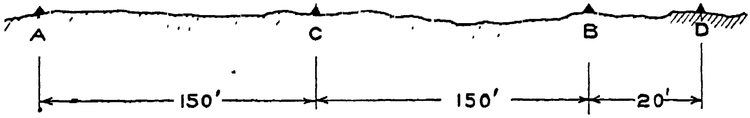


Fig. 76.

Adjustment. When used merely as an angle-measurer, the following adjustments should be tested and, if necessary, corrected:

1st. *To ascertain if the bubble tubes are perpendicular to the vertical axis of the instrument.*

To test, attach the instrument to the tripod and "set up" firmly on solid ground, preferably shaded from sun and wind. Revolve the transit upon its vertical axis so as to bring the bubble tubes parallel to a pair of diagonally opposite leveling screws. Bring the bubble of one of the tubes to the center by means of these screws. Do the same with the second bubble tube. Adjusting the second tube will throw the first one out, but repeat the alternate operations until each bubble stands in the center of its tube. Now revolve the instrument upon its vertical axis through 180°, and note if the bubble of each tube still stands in the center. If so, the tubes are in adjustment.

If the bubble of either tube runs to one end, bring it half-way back to the center by raising the opposite end of the tube by means of the capstan-headed screw. Relevel the instrument by the leveling screws, and again test the tubes. Repeat the operation, until the bubbles stand in the centers of the tubes in all positions of the instrument. It is advisable to carry out this adjustment as precisely as possible, as it will facilitate the remaining adjustments. If after several trials, it is found impossible to adjust the bubbles to the centers of the tubes, either the vertical axis is bent or the plates are sprung, and the instrument should be sent to the maker for correction. If one tube adjusts and the other does not, the fault is in the tube, and a new one should be ordered.

2d. *To make the line of collimation revolve in a plane, or, in other words, to make the line of collimation perpendicular to the horizontal axis of the telescope.*

To test, having made the first adjustment, level the instrument carefully and clamp the upper limb. Drive a stake into the ground about 300 feet *ahead* of the instrument, and drive a tack in the head of the stake. By means of the lower motion revolve the instrument on its vertical axis until the intersection of the cross-hairs approximately covers the tack. Now clamp the lower motion, and carefully adjust the line of sight upon the tack by means of or the lower limb, transit the telescope, that is, revolve it vertically, and sight to a tack in the head of a stake driven into the ground about 300 feet *behind* the instrument. Carefully adjust the tack to the intersection of the cross-hairs. Now unclamp the lower motion, and revolve the instrument upon its vertical axis until the intersection of the cross-hairs again covers the tack in the first stake. Clamp the lower motion, adjust the line of sight carefully by means of the tangent screw, again transit the telescope, and sight in the direction of the second stake. If the intersection of the cross-hairs falls upon the tack in the second stake, the line of collimation is in adjustment. If it does not, it will have to be adjusted.

In Fig. 74, A is the position of the instrument, and B is the forward stake. If the instrument is in adjustment, the line of sight after transiting the telescope and revolving upon the vertical axis should strike the point B′. If the instrument is not in adjustment, the line of sight after transiting the telescope will in the first instance strike some point as C. Drive a stake at this point and carefully center a tack. After revolving the instrument upon its vertical axis and again transiting the telescope, the line of sight will fall at a point C′ as far on one side of B′ as C was on the other. Drive a stake at C′ and carefully center it. Carefully measure the distance C C′; and center a stake at B′, half-way between the two points. Now, by means of the capstan-headed screws attached to the diaphragm carrying the cross-hairs, move the cross-hairs until their intersection covers the point B″, midway between B′ and C′. Now repeat the operation of testing the adjustment and correcting the position of the line of collimation, until the points B and B′ are in the same straight line.

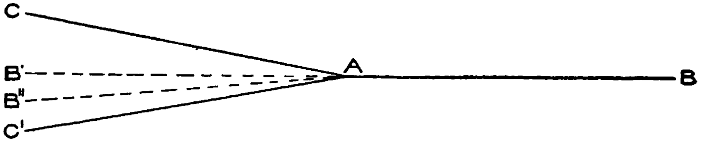


Fig. 74.

It is necessary only that the line of collimation shall be as an angle-measurer it is desirable that the vertical cross-hair be at right angles to the horizontal axis of the telescope when the instrument is level.

To test this, set up the instrument at some convenient point, 200 or 300 feet from a wall, tree, or other convenient object, upon which a point is clearly defined by a tack or otherwise. Carefully level the instrument, and cover this point accurately with the lower extremity of the vertical hair. Clamp the horizontal axis of the telescope; and by means of the tangent-screw slowly move the telescope in a vertical plane, and note if the hair continues to cover the point from one extremity to the other. If it does, the hair is in its proper position. If not, loosen the diaphragm screws and turn the diaphragm vertically until the hair covers the point from end to end. This adjustment will disturb the last one and the two must be tested and corrected alternately until in perfect adjustment.

If the transit is to be used for leveling, it is necessary that the *horizontal* cross-hair be in the optical center of the object glass.

To test, set the instrument up firmly 200 or 300 feet from a wall, tree, or other convenient object, and, after leveling, carefully center the intersection of the cross-hairs upon a well-defined point. Clamp the axis of the telescope, turn the instrument upon its vertical axis through 180°, and carefully center a point upon the intersection of the cross-hairs in this new position. Clamp the vertical axis, unclamp the telescope axis, transit the telescope, and carefully center the intersection of the cross-hairs upon the first point. Now clamp the telescope, loosen the vertical axis, and again revolve the instrument through 180°. If the line of collimation again strikes the second point, the horizontal cross-hair is in adjustment. If not, carefully center this third point, bisect the distance between the second and third points, and move the cross-hair diaphragm until the intersection of the cross-hairs covers this fourth point. This adjustment will disturb the last two, and the three must be repeated in succession until accurately adjusted.

3d. *To make the horizontal axis of the telescope perpendicular to the vertical axis of the instrument.*

To test, set up the instrument as explained for the last adjustment, and, after leveling, center carefully a point at each extremity axis and transit the telescope, bringing one extremity of the horizontal cross-hair upon one of the points previously established. If the other extremity coincides with the second point, the axis of the telescope is in adjustment.

If the axis is out of adjustment, the method of procedure is best illustrated by Fig. 75.

A A′ is the line covered between the extremities of the horizontal wire or hair when the axis of the telescope is in adjustment. If it is not in adjustment, the wire will in the first position of the telescope cover the line A B, and in the second position the line A B′. Therefore bisect the distance B B′, and raise or lower the adjustable end of the telescope axis until the wire covers A A′. Now repeat the test, and the correction if necessary.

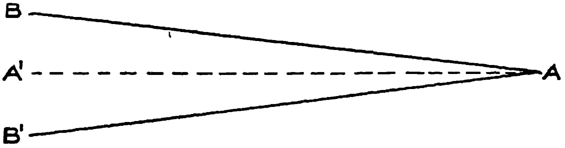


Fig. 75.

4th. *To make the axis of the telescope bubble tube parallel to the line of collimation of the telescope.*

This adjustment should be tested and corrected by the "peg" method as follows: Select a piece of comparatively level ground, drive a stake, "set up" the transit over it, and carefully level by the *plate* levels. Next drive two stakes into the ground, one in front of the transit and the other at the same distance behind it. In Fig. 76, C is the position of the transit, and A and B are two stakes, each 150 feet from C. D is a fourth stake behind B and in line with it from C. The transit being leveled by the plate levels, bring the bubble of the telescope tube to the center of the tube, by means of the tangent-screw attached to the horizontal axis of the

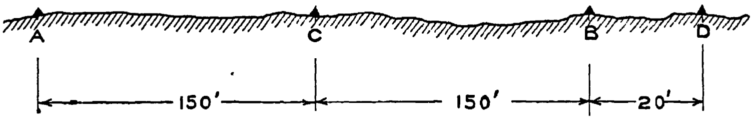


Fig. 76.

horizontal cross-hair, and note the reading. Unclamp the lower motion, turn the transit upon its vertical axis, and note the reading of a rod held upon B. The difference of the readings of the rod held upon the two points will give the true difference of level, no matter how much the telescope level may be out of adjustment.

Now take up the transit and remove it to the point D. Carefully level the transit by the plate levels, and again bring the bubble of the telescope tube to its center. Hold the rod upon the point B and note its reading. Do the same at the point A, and take the difference of the two readings. If the telescope level is in adjustment, this difference will be the same as found when the instrument was over the point C. Otherwise the tube is out of adjustment and will be corrected as follows:

Let x represent the difference of level of A and B when the transit is at C.

Let y represent the difference of level of A and B when the transit is at D.

Let z represent the difference between x and y.

If y is greater than x, subtract z from the rod reading upon A for the transit at D, and set the target at this new reading. Revolve the telescope upon its horizontal axis, by means of the tangent-screw, until the horizontal wire accurately bisects the target. Now clamp the telescope axis, and bring the bubble to the center of the tube by means of the capstan-headed screw at one end of the tube. Again hold the rod upon B, and then upon A, and take the difference of their readings. If this difference now agrees with the true difference of elevation of the two points, the adjustment is complete. Repeat the operation as often as may be necessary.

If y is less than x, add z to the rod reading upon A for the transit at D, and set the target at this new reading. Bisect the target by the horizontal cross-hair as before, clamp the horizontal axis, and bring the bubble to the center of the tube. Test and repeat as described before.

Some transits are provided with an adjustable vernier to the vertical circle or arc, which should read 0° when the telescope is horizontal. The former adjustment having been completed, the

The cross-hair intersection should be in the center of the field of vision of the eye-piece, and this adjustment may be made by means of the capstan-head screws attached to the eye-piece tube.

To "set up" the transit. Lift the instrument out of the box by placing the hands underneath the plates. Avoid lifting it by the telescope or the standards. In attaching it to the tripod be careful that the threads engage properly, and screw it down firmly. Examine the tripod legs, and see that they are properly attached to the tripod head, neither too tight nor too loose. See that the tripod shoes are tight, and, before taking up the instrument, lightly clamp all the movable parts to prevent unnecessary wear and straining. Carry the instrument in the most convenient way, taking care not to hit it against trees, lamp-posts, doors, etc.

To center the transit over a stake, rest one leg of the tripod upon the ground, and, grasping the other legs, pull the instrument in the proper direction to cover the stake. Now attach the plumb-line, and after bringing it to rest as close to the top of the stake as possible, note if the point is directly over the point in the stake. If it is not too far off the center, it may be brought closer by forcing the opposite leg into the ground or by a further spreading of the legs. After the instrument has been approximately centered, it may be accurately adjusted by means of the shifting head. The operation of "setting up" is difficult of description, and facility can be attained only by practice. Avoid having the plates too much out of level, as this will result in unnecessary straining of the leveling screws and plates.

Having centered the instrument over the stake, level it up by the levels upon the horizontal plate. To do this, turn the instrument upon its vertical axis until the bubble tubes are parallel to a pair of diagonally opposite plate-screws. Now, as you stand facing the instrument, grasp the screws between the thumb and forefinger, and turn the thumb of the left hand in the direction the bubble must move. Turn both thumbs in, or both thumbs out. Adjusting one tube will disturb the other, but adjust each alternately until the bubble of each remains in the center.

To measure a horizontal angle by means of the transit.

the north to the right through 360°, the azimuths for the preceding case will be as follows, as illustrated by the diagram:

Since the bearing of the first line A B (Fig. 79), given by the compass, is S 85° E, its azimuth will evidently be the difference between 180° and 85°, or 95°. Since the second line B C deflects to the right by 31° 30′, its azimuth will evidently be the sum of the angles 95° and 31° 30′, or 126° 30′. Since the third line C D deflects to the left 51° 0′, its azimuth will be the difference of the angles 126° 30′ and 51° 0′, or 75° 30′. The fourth line D E deflects to the right 52° 0′, and therefore its azimuth will be the sum of the angles 75° 30′ and 52° 0′, = 127° 30′.

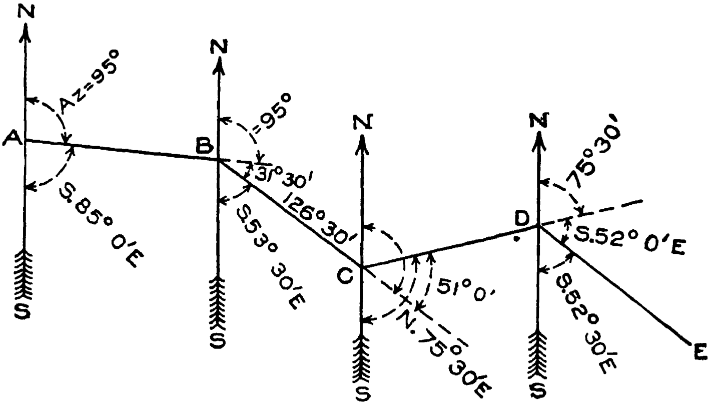


Fig. 79.

The same diagram may serve to illustrate the method of deducing the bearings of a series of lines from the deflection angles. Since the bearing of the first line as given by the compass is S 85° E, and the second line deflects to the right 31° 30′, it is evident that this second line decreases its easting by that amount, so that its bearing will be the difference between 85° and 31° 30′, = S 53° 30′ E. Since the third course deflects to the left by 51° 0′, its bearing to the east will be increased by this amount, but will pass into the northeast quadrant by 14° 30′, making its bearing 90°—14° 30′, = N 75° 30′ E. The fourth course deflects to the right 52° 0′, returning to the southeast quadrant by 37° 30′, making its bearing 90° — 37° 30′, = S 52° 30′ E.

Traversing. This is a method of observing and recording the directions of a series of lines of a survey, so as to read off, upon the horizontal circle, the angles that the lines make with some other line of the survey, which may be either a true meridian or some line adopted as a meridian for that survey.

Before starting out upon a traverse, it is best to lay out upon the ground a true meridian, either from observations on Polaris or by means of the "Solar transit," as will be explained later on. This line should be 300 or 400 feet in length, clearly defined by stakes carefully centered, one of the stakes preferably being the first station of the survey. The transit can now be set over this stake, and the line of sight carefully centered upon the second stake by means of the lower motion, the verniers first having been set to read 0°. The subsequent operations can best be illustrated by a diagram.

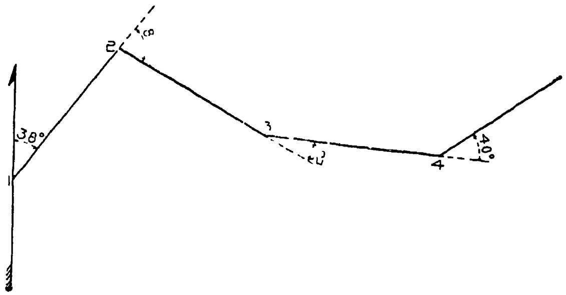


Fig. 80.

First lay out a meridian through station 1 (Fig. 80), and define it by a stake driven 300 or 400 feet away towards N. Both of these stakes should be carefully "witnessed," that they may be recovered at any time. To begin the survey, carefully center the transit over station 1, with verniers set to zero; turn the instrument upon its lower motion until the line of sight approximately covers the stake at the north end of the meridian, and carefully center it by the lower tangent-screw; lower the compass needle (if there is one), and note and record the magnetic declination. Next, with the lower motion securely clamped, unclamp the upper motion, and revolve the upper plate in the direction of station 2. Clamp the upper motion, and carefully center the line of sight by the upper tangent-screw. Note and record the angle upon the plate, which will be the azimuth of the line 1 2. Measure the distance from 1 to 2, and note and record the compass bearing as a check.

Now, with the upper motion securely clamped, remove the transit to station 2; and carefully center it over the stake with the north end of the plate ahead, that is, in the direction from 1 to 2. Transit the telescope, unclamp the lower motion, and bring the line of sight to cover station 1. Carefully center it by the lower tangent-screw. Clamp the lower motion, transit the telescope, unclamp the upper motion, and revolve the upper plate until the line of sight falls upon station 3, carefully centering it by the upper tangent-screw. Read and record the plate angle, which will be the azimuth of the line 2 3. Measure the distance from 2 to 3, and read the bearing of the needle for a check.

Now see that the upper plate is securely clamped, move the instrument to station 3, and proceed as before; and so on throughout the traverse.

In the above example (Fig. 80), since the first line is in the northeast quadrant, its bearing N. 38° E. will be the same as its azimuth (38°), and this angle is recorded upon the plate. The second line, however, passed into the southeast quadrant, and its azimuth will be recorded upon the plate as 38° + 81°, or 119°. Its bearing, however, will be S. 61° E. The azimuth of the third line will be 38° + 81° − 23°, or 96°. Its bearing will be S. 84° E. In a similar manner, the bearing of the fourth line may be deduced from the azimuth as shown upon the plate.

Fig 81 illustrates a traverse beginning in the southeast quadrant. The bearing of the first line is S. 41° E., and therefore its azimuth will be 180° − 41° = 139°. The bearing of the second line will be S. 73° E., and its azimuth will be 107°; and so on with the remaining courses of the traverse.

By a similar process of reasoning, the azimuths of courses in the southwest and northwest quadrants may be deduced from the bearings, and *vice versâ.*

We therefore establish the following rules:

(*a*) Courses in northeast quadrant—Azimuth = Bearing.
(*b*) Courses in southeast quadrant—Azimuth = Supplement of Bearing.

(*c*) Courses in southwest quadrant—Azimuth = 180° + Bearing.
(*d*) Courses in northwest quadrant—Azimuth = 360° − Bearing.

Bearing	north:	Azimuth	= 0° or 360°.
"	east:	"	= 90°.
"	south:	"	= 180°.
"	west:	"	= 270°.

For convenience in determining azimuths, the inside graduations of the horizontal circle may be numbered from 0° to 360° to the right, beginning at the *north* end; and the outside graduations from 0° to 360° to the right, beginning at the *south* end. Then keeping the north end of the plate ahead—that is, in the direction of the traverse—for a traverse beginning in either the northeast or northwest quadrants, azimuths will be recorded directly upon the plate from the *inside* graduations; and for a traverse beginning in either the southeast or southwest quadrants, azimuths will be recorded directly upon the plate from the outside graduations.

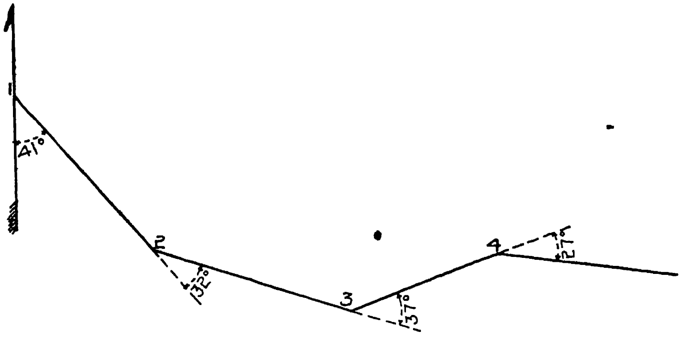


Fig. 81.

Traversing is particularly adapted to surveying roads, streets, railroads, shores of lakes, river banks, etc.; and in land surveying it possesses an advantage over the method by interior angles, on account of the readiness it affords in obtaining the bearings from the azimuths, and the greater rapidity with which the work may be platted, since the angle which each line makes with the assumed meridian or reference line, may be taken at once from the field notes.

In United States government surveys, when a traverse is run to mark the divisions between private estates and bodies of water

Keeping Notes. All notes of measurements of angles and distances should be recorded *as soon as made,* in a special notebook adapted to the purpose. Avoid the practice of making notes upon scraps of paper and in small pocket notebooks and of filling in details from memory. The notes will probably be used by other persons unfamiliar with the locality, for platting and for general information, and these persons must depend entirely upon what is recorded, and how it is recorded, for their interpretation. To this end the notes should be *clear* and *concise,* yet full enough to give all necessary information. They should permit of only one interpretation, and that the correct one.

The note keeper should bear in mind constantly the nature of the survey and the object to be attained, and this will enable him to determine what measurements are necessary. Do not crowd the notes. Use the left-hand page of the book for notes, and the right-hand page for such sketches and remarks as may be necessary. The record is usually made with a pencil, using a medium hard grade. If incorrect entries are made, erase them neatly; but avoid errors as much as possible, as too many erasures tend to discredit the work. After each notebook is filled, label it with the subject of the survey, the dates between which it was recorded, and all other information that may be of service in filing for future reference. Above all, do not lose a notebook, as it may contain information that cannot be recovered at any price.

Referring to Fig. 80, the notes would be entered in the record as follows:

Station	Distances	Deflections		Azimuth	Needle
		Right	Left		
1	220 00′	38° 00′		38° 00′	N. 38° 00′ E.
2	225 00′	81° 00′		119° 00′	S. 61° 00′ E
3	235 00′		23° 00′	96° 00′	S. 84° 00′ E.
4	190 00′		40° 00′	56° 00′	N. 56° 00′ E

Figs. 82 and 83 illustrate the method of keeping notes by means of sketches.

Checking the Traverse. For an ordinary survey not involving unusual precision, a transit reading to single minutes will be sufficient. A single measurement will ordinarily give the angle with sufficient accuracy; but should a check upon the measurement be necessary the angle may be "repeated" as explained on page 109.

As a further check against errors in angles, the magnetic bearing of each line should be read, showing the approximate directions of the lines, and by comparison with the azimuths or the deduced bearings, will serve to point out gross errors, as for instance reading right for left when measuring deflection angles. This check upon the angles should always be applied *in the field*, so that errors may be rectified before leaving the work.

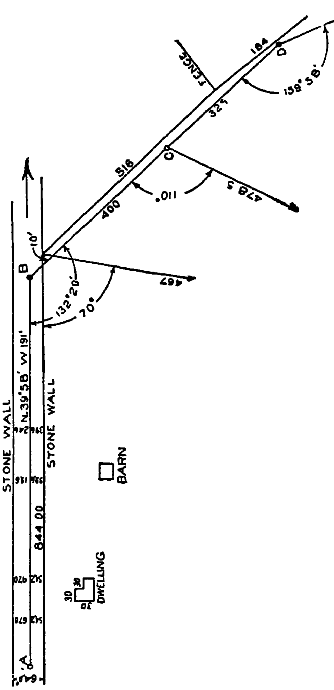


Fig. 82.

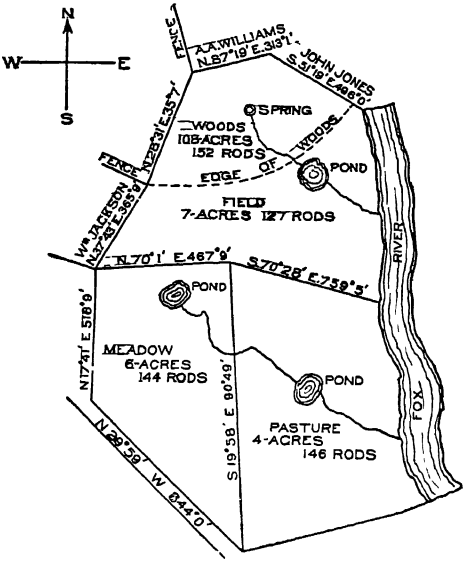


Fig. 83.

If the traverse involves a closed area, the accuracy of the transit work may be tested by adding together all of the measured angles. The sum of the interior angles should equal $(n-2) \times 180°$, n being the number of sides of the field. For the deflection angles, the sum of all the deflections to the *right* should differ from the sum of all the deflections to the left by 360°; that is to say, *the algebraic sum of the deflection angles should be* 360°.

It is sometimes desirable to check the lengths of the courses of the measurement of any line, it should be measured, preferably in the opposite direction. In a closed traverse, it is well to run diagonal lines across the traverse as an additional check upon both the angles and the measured distances.

For city work, the engineer should lay out a true meridian 300 or 400 feet long, and mark the extremities of the line by permanent monuments set in the ground and carefully protected from disturbance. To do this, in some convenient place permitting an unobstructed line of sight, drive a large stake, and mark its center by a hollow-headed tack. Center the transit carefully over this point, and proceed to lay out a true meridian, preferably with the solar attachment. Mark the direction of this line by a second stake carefully centered by a tack as before. Now, about 25 feet from the first stake, and in line with it and the second stake, excavate a hole in the ground about three feet in diameter and five feet deep, or deep enough to be below the frost line. Next build a foundation of concrete about two feet square and three feet deep. Before this has "set," insert in it a cut-stone post about nine inches square at its lower end and of such length that its top will come just below the surface of the ground, and having set into its top a copper bolt about $\frac{3}{4}$ inch by 4 inches. The post may be centered in the concrete by the transit, and should be set "plumb."

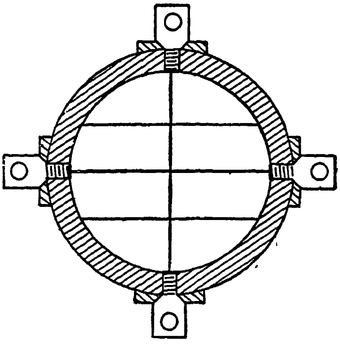


Fig. 84.

Now locate and build a second monument in line with the second stake and a few feet from it. After the concrete has set firmly, again set the transit carefully over the center of the first stake, and accurately align it by the tack in the second stake. Now "plunge" (reverse) the telescope, and carefully center a point in the top of the copper bolt; mark this point with a steel punch. In the same way center a point in the top of the bolt of the second monument. The monuments may be protected by enclosing them in cast-iron valve-boxes with covers. Either one or both of these monuments may be used as "standard" bench-marks from which all the levels and grades may be ascertained. For this purpose a city datum may be assumed, or better, the bench-mark may be connected by a line of levels with a bench-mark of the U. S. Coast and Geodetic Survey, or, if such is not available, with a bench-mark of the nearest railroad.

THE STADIA.

Attached to the diaphragm carrying the horizontal and vertical hairs, are two auxiliary horizontal hairs called "stadia" hairs or wires. These hairs may be either fixed in position or adjustable, but the fixed hairs are the better for field use and cost much less. See Fig. 84. Any instrument-maker will equip a level or a transit with either fixed or adjustable stadia wires. and they should be included in every outfit.

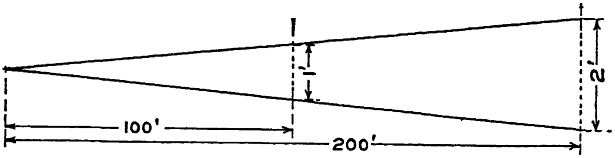


Fig. 85.

The **stadia** is used for measuring horizontal distances and differences of elevation, without the use of chain or tape or other apparatus except the leveling rod or a specially graduated stadia rod. It is based upon the principle of the similarity of triangles. Thus, if the stadia hairs are spaced so as to intercept one foot upon a rod held at a distance of one hundred feet, the rod intercept for any other distance will be in direct proportion to the first.

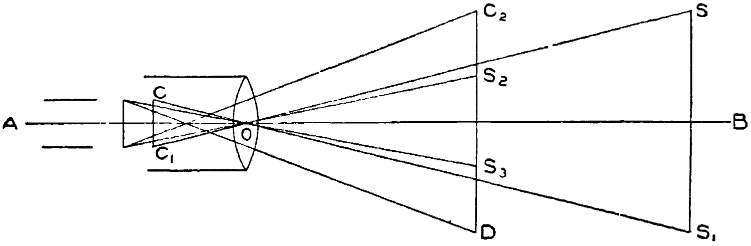


Fig. 86.

Unfortunately the construction of the telescope of an engineering instrument modifies the above simple statement, and a formula for the use of the stadia will now be deduced.

Let O in Fig. 86 be the optical center of the object-glass of the telescope. This point may be assumed at the center of the lens, and the error involved in such assumption is inappreciable and may be neglected.

Let SS_1 be a portion of the stadia rod covered by the stadia hairs CC_1. From C and C_1 draw the lines CS_1 and C_1S through the optical center of the object-glass. Upon looking through the eyepiece of the telescope, C will be seen as at S_1 and C_1 as at S.

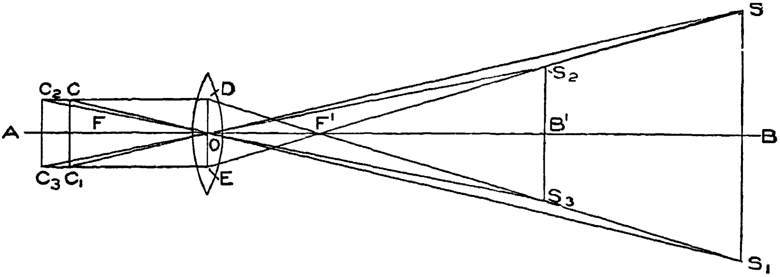


Fig. 87.

Call i the distance between the stadia hairs, s the intercept upon the rod, f' the distance from O to the wires, and d the distance of the rod from O.

The triangles COC_1 and SOS_1 are similar, and therefore we have the proportion

$$i : s :: f'' : d$$

$$\text{therefore } d = \frac{f's}{i}. \qquad (1)$$

But f' varies with d. That is to say, if the rod were to be moved closer to the instrument, as at C_2D, the lens would be moved farther from the wires, or the wires from the lens, and in either case the wire interval would intercept a shorter space upon the rod, as S_2S_3. The ratio $\frac{f'}{i}$, or its equal $\frac{d}{s}$, will therefore vary for each position of the rod. But however they vary, we have from a well-known principle of optics:

$$\frac{1}{f'} + \frac{1}{d} = \frac{1}{f}, \qquad (2)$$

in which f is the principal focal distance of the lens, and f' and d are any pair of conjugate focal distances. Substituting the value of $\frac{1}{f'}$ from (1) in (2), there results the equation

$$d = \frac{f}{i} s + f. \qquad (3)$$

Equation 3 gives the distance of the rod from the lens.

We can establish some very important relations:

In Fig. 87 lay off $OF' = OF =$ principal focal distance of lens $= f$.

C and C_1 being the stadia wires, draw C D and C_1 E parallel to the axis of the lens, and through F′ draw D S_1 and E S; then will S S_1 = the intercept upon the rod.

The distance of the rod from the point F′ is

$$d' = d - f = (\frac{f}{i} s + f) - f = \frac{f}{i} s.$$

From the similarity of the triangles EF′O, SF′B and S_2F′B′, we have:

$$F'O : OE :: F'B : BS :: F'B' : B'S_2.$$

Therefore the points S, S_2, F′, and E lie in the same straight line; and therefore, wherever the cross-hairs are situated, or, more strictly, whatever may be the position of the lens, the visual lines defined by the stadia wires will intercept the elements of the cone of light defined by SF′S_1.

All distances must be measured from the center of the instrument; and therefore to the expression $d = \frac{f}{i} s + f$ must be added a quantity that will represent the distance from the center of the lens to the plumb-line. This quantity is variable, of course, but an average value is usually taken. Call this quantity c. The quantity f may be found by focusing the instrument on a star, and measuring the distance from the center of the lens to the cross-hairs. We can therefore determine the quantity $f + c$. This, however, is usually supplied by the instrument-maker, and with greater precision.

The complete equation, therefore, for any distance measured with the stadia, is:

$$d = \frac{f}{i}s + (f + c) \qquad (4)$$

If, then, upon level ground we lay off the distance $f + c$ in front of the plumb-line, drive a stake, measure from this stake a distance of 100 or 200 feet, or any other convenient distance, and note the rod intercept, then in the formula $d' = \frac{f}{i}s$, d' and s are measured, and we can determine the ratio $\frac{f}{i}$; or, if f has been previously determined, we can determine the value i or the distance between the cross-hairs.

The ratio $\frac{f}{i}$ being known, distances can be found from equation 4. It is usually most convenient to make this ratio 100, so that at a distance of 100 feet the wires will intercept one foot upon the rod.

The rod may be either an ordinary leveling rod, or a stadia rod divided specially for the telescope and wires. When the rod is specially graduated, it may be in either one of two ways. Either it may be graduated so as to give the distance from F′, in which case the quantity $f + c$ will have to be added in each instance; or it may be graduated to give distances from the center directly.

If the rod is to be graduated specially, proceed as follows:

Carefully level the line of collimation of the telescope, and lay off from the plumb-line the distance $f + c$. From the point thus established measure off any convenient distance, as 500 feet, on a horizontal plane. Set up the rod, not yet graduated, at this point, and hold it carefully perpendicular to the line of sight from the telescope. This can best be done by means of a plumb-line. Be careful to eliminate all parallactic motion of the wires or the rod, when the eye is moved up and down before the eye-piece.

Mark on the rod very carefully the apparent place of the lower wire. This should be about one-quarter the length of the rod from one end if the horizontal distance first laid off is about one-half the greatest distance for which the rod can be used. The middle wire will then be at about the middle of the rod, and the upper one at about one-quarter the length of the rod from the other end.

Mark the latter point carefully. The wire interval for a space of 500 feet from F′ has thus been found. One-fifth of this space will be the wire intercept at a distance of 100 feet; twice the space, the intercept for 1,000 feet; and so on. The intermediate spaces can thus be graduated. It must not be forgotten that in using the rod thus graduated, the quantity $f + c$ must be added to the distance indicated by the rod, to reduce the distance to the center of the instrument.

If the rod is to be graduated to give distances from the center of the instrument directly, proceed as before, marking the spaces upon the rod corresponding to the distances measured upon the ground. The quantity $f + c$ will not now have to be added to the distances given by the rod; but for every point other than that for which the rod is graduated, the distance will be in error by some fractional part of $f + c$. The reason for this will be apparent by referring to Fig. 87. If the distance is less than that for which the rod was graduated, the rod readings will indicate too small a distance; and for a distance greater than the standard, the rod readings will indicate a distance too great. It is therefore more exact to mark the wire interval at 100 feet, 200 feet, and so on through the length of the rod. Each space thus determined can be divided up as desired; and the error involved in any reading will then be much smaller than if the rod were graduated for a

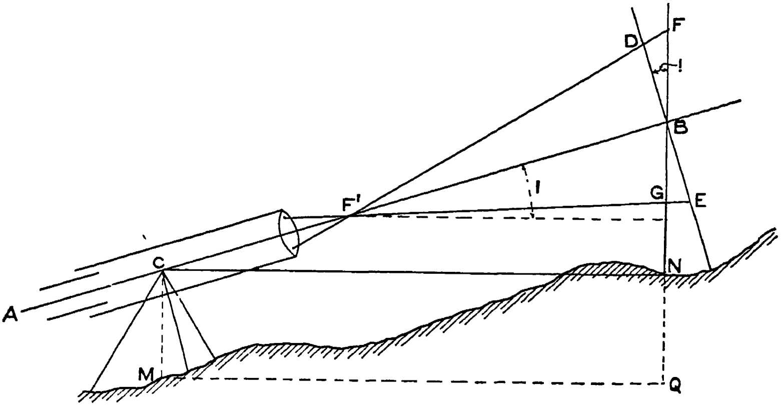


Fig. 88.

Thus far the rod has been assumed as held perpendicular to the line of sight, which of course will always be the case when using the stadia in the leveling instrument. The stadia, however, finds its greatest usefulness in connection with the transit, when the line of sight is seldom horizontal. If, at the same time the rod intercept is read, the vertical angle is noted, differences of elevation may be determined, as well as the distances.

A formula will now be deduced for reducing inclined readings to the horizontal, and for determining differences of elevation, the rod being held vertical.

In Fig. 88, let the angle of inclination of the line of sight to the horizontal plane be called B C N = F B D = I. This angle will be measured upon the vertical circle of the transit. If the rod be held perpendicular to the line of sight, the intercept upon the rod = D E = s. Represent the rod intercept when the rod is held vertical by s'. Now since the angle F D B = 90° nearly, $DE = FG \cos I$, or $s = s' \cos I$. But $CB = \frac{f}{i} s + (f + c) = \frac{f}{i} s' \cos I + (f + c)$. Therefore the horizontal distance to the rod $= CB \cos I = \frac{f}{i} s' \cos^2 I + (f + c) \cos I = CN$. The vertical distance of the point B above the horizontal plane through the axis of the telescope $= BN = CB \sin I = \frac{f}{i} s' \cos I \sin I + (f + c) \sin I = \frac{1}{2} \frac{f}{i} s' \sin 2I + (f + c) \sin I$.

For vertical angles less than 5° the quantity $(f + c) \sin I$ is less than $0.1\,(f + c)$ and may be neglected.

The Use of the Stadia in the Field. In using the stadia wires in level country, no special instructions are necessary, as the line of sight is at all times horizontal. Over very uneven ground, the use of the level and stadia is very limited. However, there are often conditions in which the stadia wires in a leveling instrument are a very great convenience. The range of the instrument may sometimes be increased by using the center wire together with one of the stadia wires, but the instrument should be carefully tested to ascertain if the stadia wires are equally spaced with reference to the middle wire.

For extended surveys over uneven country, the transit a stadia are particularly adapted, and especially for filling in details of an extended topographical survey. The saving of time and expense are important elements in favor of the transit and stadia as compared with the transit and tape; and with a little practice and attention to details the results should be fully as accurate. Certainly, when an engineer must depend upon unskilled help to carry the tape, there can be no choice as to which to use.

For use with the stadia, the transit should be provided with a complete vertical circle, reading to minutes at least; and a level tube should be attached to the telescope. The eye-piece should be inverting. Before starting out upon a survey, the transit should be carefully tested and corrected through all of its adjustments. The field operations are then as follows:

Set up the instrument over a principal station of the survey, and level it carefully. If a solar attachment is available, it will be desirable to lay out a true meridian, from which the declination of the needle may be determined. Now determine the height of the cross-hairs by holding the stadia rod close to the side of the instrument, and noting the height of the center of the horizontal axis of the telescope. Locate the second station carefully, and turn the telescope upon the horizontal axis until the center wire cuts the division upon the rod (held upon the ground) representing the height of the axis of the instrument above the ground at the first station. Now determine the azimuth of the line connecting the two stations, read the vertical angle of the telescope, and determine the rod intercept. Enter these items in the field book and proceed to take observations upon sub-stations (called "side-shots").

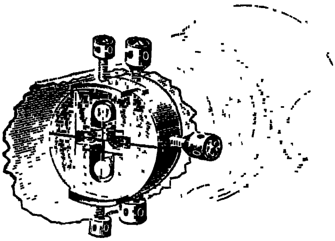


Fig. 89.

The same program is to be repeated for each case, except that the side-shots may or may not be taken upon points indicated by stakes. The principal stations of a stadia survey should be permanent; the stakes should be driven, and "witnessed" so as to

Having now located all the necessary points from the first station, remove the instrument to the second station, and set it up with the north end of the plate in the direction of the survey. Having carefully leveled the instrument, determine the height of its axis as before, and send the rod back to the first station. Transit the telescope, and sight upon the rod as before. Read vertical angle and stadia rod, and determine azimuth, and these will serve to check the former determinations.

In moving from one station to another it is advisable to set the scale of the horizontal circle to zero. Transit the telescope again and locate the next station; and so on throughout the survey.

The principal stations of a stadia survey may have been located by a previous triangulation, in which case it will probably be necessary to locate intermediate stations as the survey progresses. Or all of the stations may be located during the progress of the survey. The courses connecting the principal stations form the "backbone" of the survey, and the azimuths and distances should be checked at every opportunity.

In keeping the field notes, represent the principal stations by triangles, as $\triangle_1$ $\triangle_2$ $\triangle_3$, etc.; and the secondary stations by circles, as $\odot_1$ $\odot_2$ $\odot_3$, etc.

Below will be found an example of the method of keeping notes. Use the right-hand page for sketches, or for such additional notes as may be necessary.

Sta	VER. A.	VER B.	Obs. Dist.	Cor. Dist	Vert Ang	Diff El	Elev
At △ 7	Elev. △ 7	= 633 5′	H I =	5 1′			
△ 5	152°51′	332°51′	523 2	523 2	−0°11′	− 1 6	631 9
⊙ 1	89°25′		524	520	+5°13′	+47 3	680 8
2	85°18′		355	353	+5°48′	+35 5	669 0
3	90°06′		293	290	+5°27′	+27 6	661.1
4	106°35′		235	235	+1°23′	+ 5 8	639 3
5	114°50′		245	245	+0°33′	+ 2 3	635 8
6	132°33′		223	220	−7°52′	−30 4	603.1
7	152°57′		228	202	−8°55′	−69 2	564 3
8	75°04′		277	273	+5°20′	+25 5	659.0

Stadia Rods. Telemeter or stadia rods are made of clear white pine well seasoned, about ⅞ of an inch thick, from 4 to 4½ inches wide, and from 10 to 16 feet long. They are protected by a metal shoe to keep the lower end from being battered or split. The rod is stiffened by having a piece 2½ inches wide along its back. Generally a stadia rod is hinged at the center for greater convenience in transportation, and at the same time it is provided with a bolt on the back to protect the graduation and to hold it in position when in use.

A self-reading level rod may be used for distances if the wires are adjustable (see Figs. 89 and 84), or if the wire interval has been determined in standard units. The rods used in connection with this grade of work differ from those employed in ordinary leveling. Those with graduations have the inner surface recessed to protect the graduated surface, and are painted white with the scale in black. The forms of graduation are different on different rods. In some, the unit of measure is the meter, while others have the foot, as will be described later. When telemeters are in use, they are open, laid flat, and held securely in line by the brass clip (or bolt) above referred to. They are sometimes provided with a target.

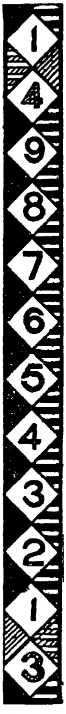


Fig. 90.

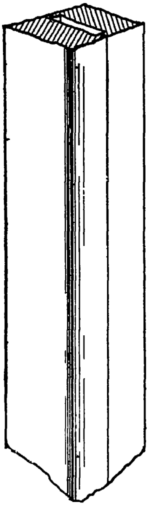


Fig. 91.

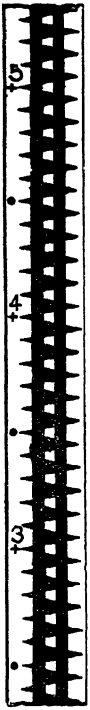
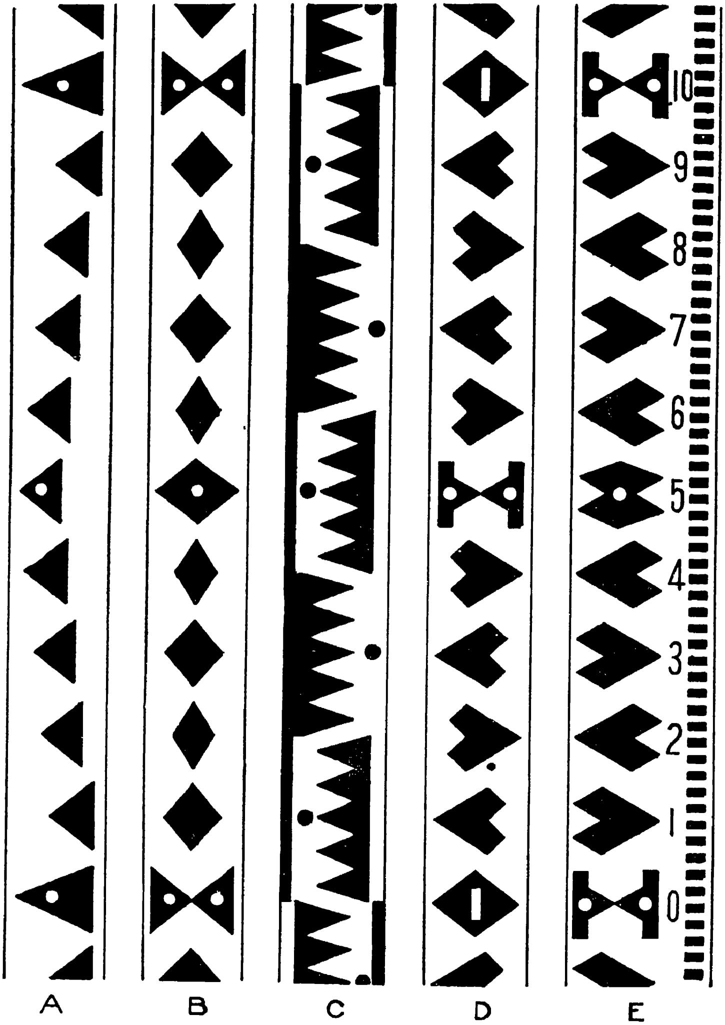


Fig. 92.

In order to have the rod held in a perfectly vertical position,



A, B and C—U. S Coast Survey. D—U S Lake Survey. E—U. S Engineers.

Fig. 93.

which the rodman can tell whether the rod is in a vertical plane.

Figs. 90 and 93, D, show two types of graduations suitable where the meter is the unit. Fig. 93, D, has for many years been used by the United States Coast and Geodetic Survey as well as by the United States Lake Survey. The angles of graduation divide the rod into two centimeter intervals. Fig. 90 shows the rod used on the survey of the Mexican border. The graduation is apparent, and no further explanation need here be given.

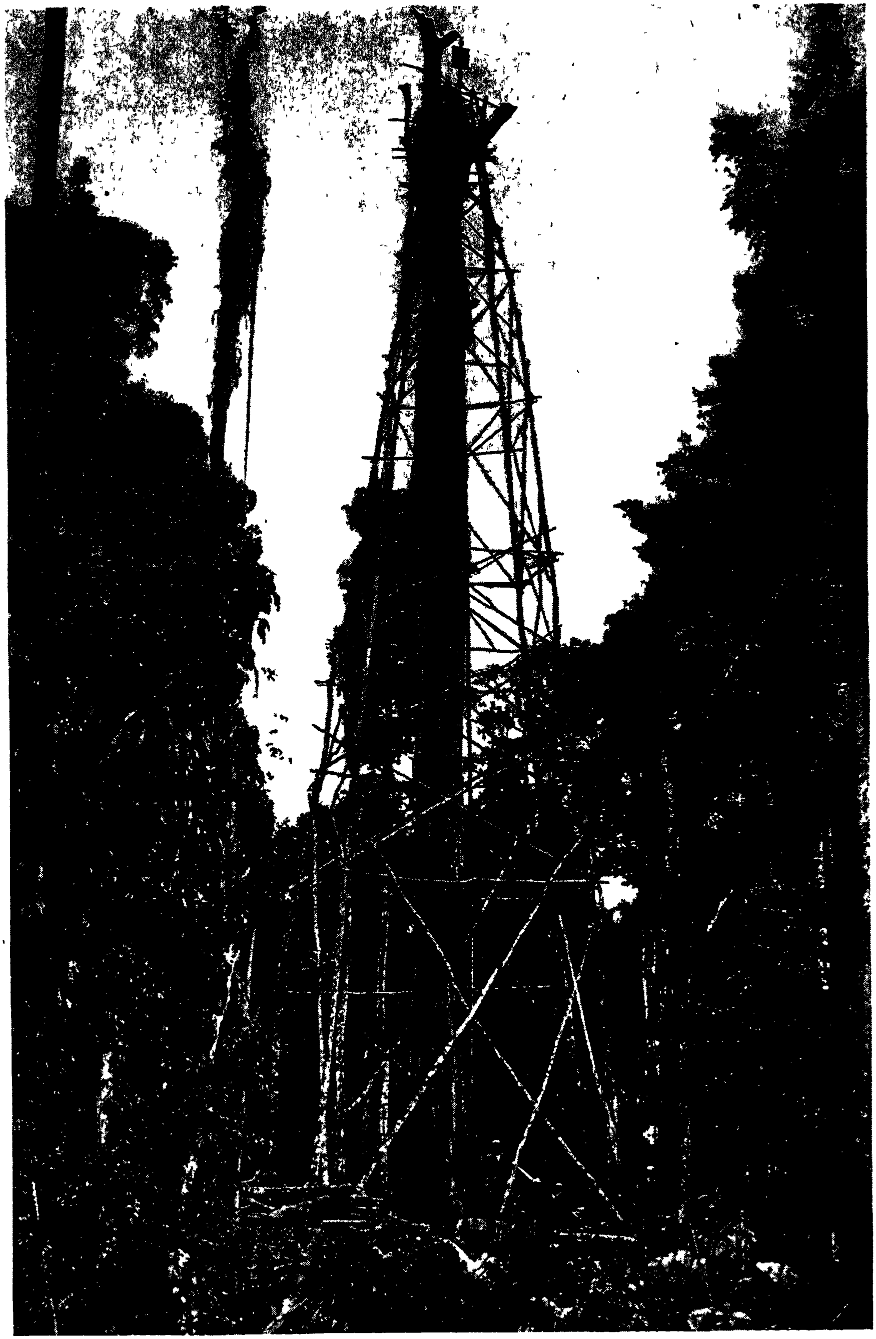
Figs. 92 and 93, C, are types suitable where the foot is the unit. In Fig. 92 the width comprised between the ends of the points divide into five equal parts, the vertical black lines taking up two of these differences. The diagonal then gives one hundredth of a foot, and permits readings direct to single feet. Fig. 91 shows a plain rod without scale, and the unit is the foot. Classes A, B, C, D and E, Fig. 93, belong to the respective surveys as indicated.

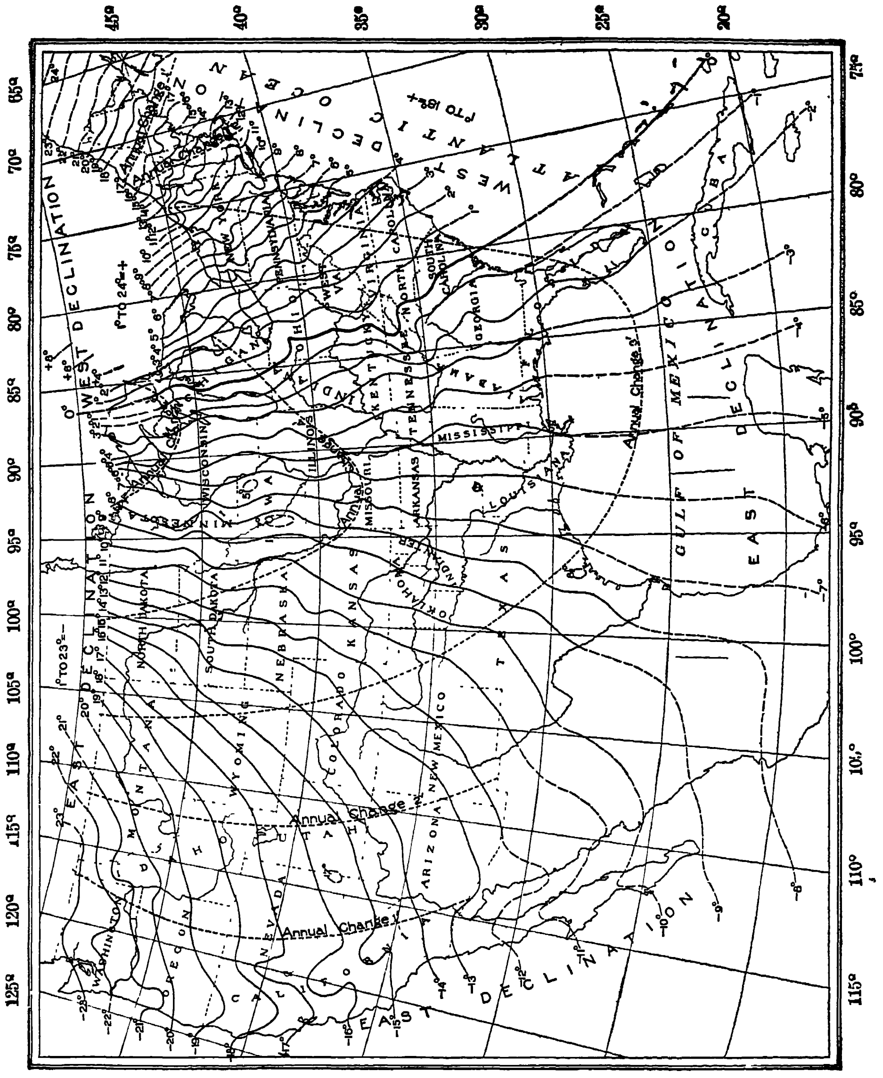
TIME OF ELONGATION AND CULMINATION OF POLARIS.

DATE IN 1899.		EAST ELONGATION.		UPPER CULMINATION.		WEST ELONGATION.		LOWER CULMINATION.	
		h.	m.	h.	m.	h.	m.	h.	m.
January	1	0	41.9	6	36.7	12	31.5	18	34.7
	15	23	42.7	5	41.7	11	36.2	17	39.4
February	1	22	35.5	4	34.3	10	29.1	16	32.3
	15	21	40.3	3	39.0	9	33.9	15	37.0
March	1	20	45.1	2	43.6	8	38.6	14	41.8
	15	19	50.0	1	48.8	7	43.5	13	46.8
April	1	18	43.0	0	41.7	6	36.5	12	39.8
	15	17	48.0	23	42.8	5	41.5	11	44.8
May	1	16	45.2	22	39.9	4	38.7	10	41.9
	15	15	50 3	21	45.0	3	43.8	9	47.0
June	1	14	43.6	20	38.4	2	37.1	8	40.4
	15	13	48.7	19	43 5	1	42.2	7	45.5
July	1	12	46.1	18	40.9	0	39.6	6	42 9
	15	11	51.2	17	46.0	23	40.8	5	48.0
August	1	10	44.7	16	39 5	22	34.3	4	41.5
	15	9	49.8	15	44 6	21	39.4	3	46.6
September	1	8	43.2	14	38 0	20	32.8	2	40 0
	15	7	48.3	13	43 1	19	37 9	1	45 1
October	1	6	45 5	12	40 3	18	35.1	0	42 3
	15	5	50.5	11	45 3	17	40.1	23	43 4
November	1	4	43.7	10	38.5	16	33.3	22	36 5
	15	3	48 5	9	43 3	15	38.1	21	41 3
December	1	2	45.5	8	40 3	14	35 1	20	38 3
	15	1	50.2	7	45 0	13	39.8	19	43.0

AZIMUTH OF POLARIS AT ELONGATION.

YEAR.	25°	30°	35°	40	45	50°	55°
1900	1° 21′ .2	1° 24′ .9	1° 29′ .8	1° 36′ .0	1° 44′ .0	1° 54′ .4	0° 08′ .3
1901	1 20 .8	1 24 .6	1 29 .4	1 35 .6	1 43 .6	1 54 .0	2 07 .8
1902	1 20 .5	1 24 .2	1 29 .0	1 35 .2	1 43 .2	1 53 .5	2 97 .2
1903	1 20 .1	1 23 .9	1 28 .7	1 34 .8	1 42 .7	1 53 .0	2 06 .6
1904	1 19 .8	1 23 .5	1 28 .3	1 34 .4	1 42 .3	1 52 .5	2 06 .1
1905	1 19 .4	1 23 .1	1 27 .9	1 34 .0	1 41 .8	1 52 .0	2 05 .6
1906	1 19 .1	1 22 .8	1 27 .5	1 33 .6	1 41 .4	1 51 .5	2 05 .0
1907	1 18 .7	1 22 4	1 27 .1	1 33 .2	1 40 .9	1 51 .0	2 04 .4
1908	1 19 4	1 22 1	1 26 .8	1 32 .8	1 40 .5	1 50 .6	2 03 .9

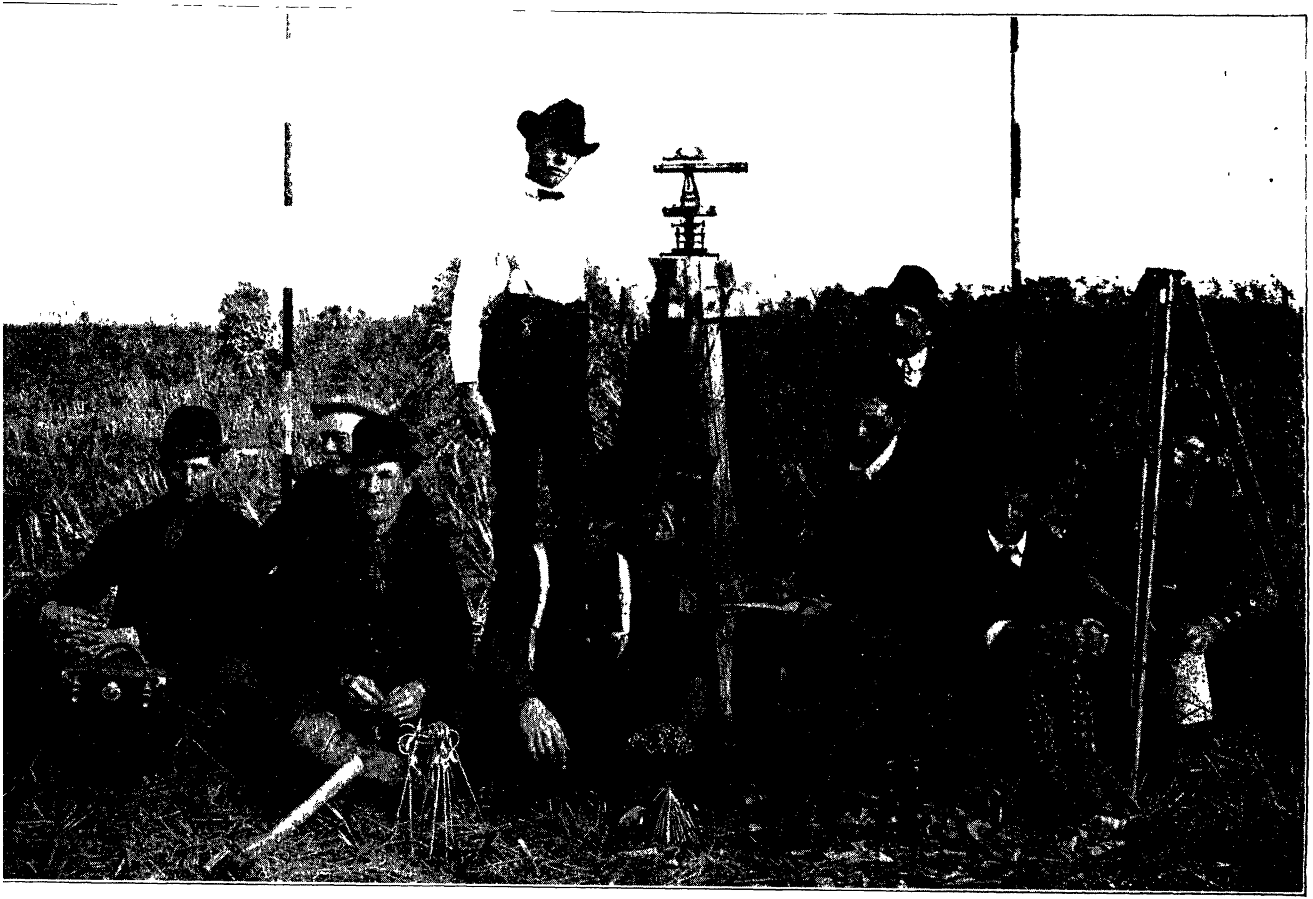




UNITED STATES GEODETIC SURVEY

Base Map of the United States

Declinations west of the line of zero declination are —, those east are +.



TYPICAL SURVEYING PARTY AT TRIANGULATION STATION—READY FOR WORK

Courtesy of R. Isham Randolph, Chicago, Illinois

PLANE SURVEYING.

PART III.

THE GRADIENTER.

The vertical circle or arc of the transit or theodolite, under ordinary circumstances, furnishes the means of measuring the vertical angle through which the line of collimation is turned, or, on the other hand, of turning the line of collimation through any desired vertical angle. Much of the work of the engineer consists in measuring slopes or grades, or in setting a line at a certain slope or grade; and the data are given, not in terms of the vertical angle directly, but usually by the amount of rise or fall per 100 feet. Thus, a rise or fall of 2 feet in 100 feet is designated as a 2 per cent grade; a rise or fall of 50 feet to the mile would be designated as a 0.95 per cent grade, etc. The ratio of these two quantities, *rise* (or *fall*) to *reach* is evidently the natural tangent of the angle of slope; and before the vertical circle can be used for setting off such slopes, the ratio must be transformed into degrees and minutes of arc.

The tangent-screw of the horizontal axis of the telescope without the aid of the vertical circle, provides the means of quickly and accurately setting off slopes directly, when the vertical angle does not exceed fifteen or twenty degrees. For this purpose, the ordinary tangent-screw is replaced by a fine screw, with very uniform pitch and large graduated head, and also a graduated scale from which may be read the number of turns or double turns made by the screw. The graduated head fits friction-tight upon the neck of the screw, so that its index may be made to read zero when the line of collimation is horizontal; and it is usually divided into fifty parts, so that, after the number of double turns is read from the scale, it will give the number of fiftieths of a single turn, or hundredths of a double turn. (See Fig. 94.)

Let the distance of the screw from the axis about which the

the threads by t. If the screw is turned through one revolution, the lever AD (Fig. 94) is moved through the distance DE, and the line of collimation through the distance BC, upon the rod PQ. Now, the tangent of the angle DAE $= \frac{DE}{AD} = \frac{BC}{AB} = \frac{t}{l}$. To this ratio, the maker of the instrument can give any convenient value, but it is customary to make it $= \frac{1}{200}$, and it will be so considered throughout this discussion.

If, then, the line of collimation be directed toward the graduated rod PQ, the space over which the line of collimation is moved for one revolution of the screw is $\frac{1}{200}$ of the distance of the rod from the instrument; and the space upon the rod over which it is moved for two revolutions of the screw $= \frac{2}{200} = \frac{1}{100}$ of the above distance. If the screw is turned through less than a single revolution, it will be indicated upon the graduated head, as, for instance, $\frac{35}{50}$ of a single turn, the intercept upon the rod being $\frac{7}{10} \times \frac{1}{200} = \frac{7}{2,000}$ of the distance from rod to instrument—it being understood, of course, that the rod is held perpendicular to the line of collimation in its initial position. The index of the graduated head should read zero when the line of collimation is horizontal, and the reading of the scale of revolutions should be zero at the same time.

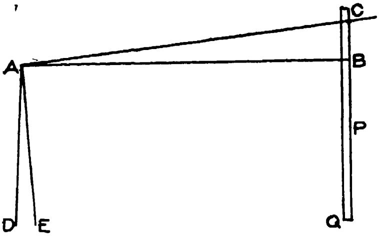


Fig. 94.

The gradienter may be used as a telemeter, as a level, or simply as a grade-measurer, as will be explained in what follows.

Call s the intercept upon the rod for any movement of the gradienter-screw, and d the distance from the instrument to the rod.

If the number of revolutions of the gradienter-screw is known, whether s and d are known or not, the tangent of the angle of inclination of the line AC is known, and the instrument is a grade-

If the space s and the number of revolutions of the gradie screw are known, the distance d is known, and the instrumen then a telemeter.

If the distance d and the number of revolutions of the gradienter-screw are known, the space s is known, and the instrument then serves the purpose of a level.

As a gradienter, the instrument may be used either to measure the grade of a given line, or to lay out a line to a required grade.

(1.) Let AB (Fig. 95) be the line whose grade is required. Set the transit up over the point A, and level carefully. Measure the height of the cross-hairs above the ground by holding the rod beside the instrument and noting the point upon the rod directly opposite the center of the horizontal axis of the telescope. Bring the line of collimation CE horizontal by means of the bubble attached to the telescope (the instrument is supposed to be in adjustment), and set both the indexes to zero. Now carry the rod to the point B, and by means of the gradienter-screw turn the telescope in a vertical plane until the line of collimation strikes the point D as far above B as C was above A. Now count the number of full turns from the reading of the scale by the screw-head, and the number of fractions of a turn from the divided head. The former will give the rise (or fall) in *feet* per 100, and the latter in hundredths of a foot. It must be remembered that if the screw has made more than a whole turn past the last number on the scale the reading of the head must be increased by fifty.

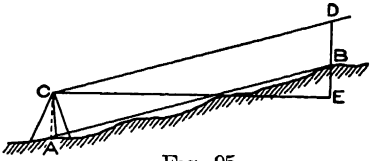


Fig. 95.

Thus, if the reading of the scale is 3 and the reading of the head is 35, plus one whole revolution, the rise (or fall) per 100 feet will be as follows:

3	*double* turns - - - - -	3.00 ft.
1	*single* turn - - - - -	0.50 ft.
$\frac{35}{50}$	" " - - - - - -	0.35 ft.
		3.85 ft.

So that the slope of CD, which equals that of AB, is therefore

EXAMPLE FOR PRACTICE.

If the scale reads 2 and the head 31, determine the percentage of slope.

(2.) It is required to lay out in a given direction a line with a given percentage of slope from the point A. See Fig. 96.

Set up the instrument on the given point, as B, and level it carefully. Measure the height of the cross-hairs above the ground as before, and set the pointers to read zero with the bubble in the center of the telescope tube. Now revolve the line of collimation in a vertical plane by means of the gradienter-screw so as to set off the required slope. For instance, suppose it is required to set off a slope of 2.78 per cent. The screw should be turned five complete revolutions as indicated upon the scale, plus $\frac{28}{50}$ of a revolution as indicated upon the divided head of the screw:

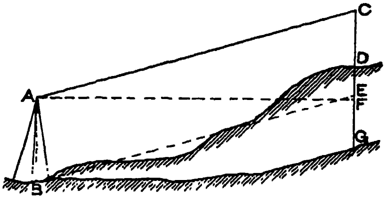


Fig. 96.

$$\frac{5}{200} \times 100 \text{ ft.} \qquad = 2.50 \text{ feet.}$$
$$\frac{28}{50} \times \frac{1}{200} \times 100 \text{ ft.} \qquad = \underline{0.28 \text{ feet}}$$
$$= 2.78 \text{ feet per 100 feet.}$$

Now carry the rod to any convenient point, as G, in the direction of the required line; hold it in a vertical position; and note the height of the line of collimation. Take the difference between this and CE (= AB). If this difference, as EG, is positive, it gives the height of the grade line *above* the ground and indicates a *fill* at the point. If the difference is negative, as DE, it gives the depth of the grade line *below* the ground and indicates a *cut*.

EXAMPLE FOR PRACTICE.

Let it be required to set off a 3.35 per cent grade, and describe the operation in detail.

When the gradienter is used as a telemeter, it may be upon level or sloping ground.

(1.) *Upon Level Ground.* Set up the transit at one end of the line, and level carefully. Bring the bubble of the telescope level to the center of its tube, and both gradienter scales to zero.

Now send the rod to the next station and let it be held vertical; adjust the target to the line of collimation and take the reading. Now turn the gradienter-screw through two revolutions and take the reading again (see Fig. 97). The difference of the two readings gives DE in feet; and since the gradienter-screw has been turned through two revolutions, CE = 100 DE. Thus, if DE = 3.25 feet, CE = 325 feet.

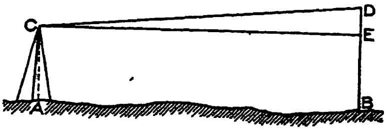


Fig. 97.

(2.) *Upon Sloping Ground.* On sloping ground the first reading upon the rod cannot be taken with the telescope horizontal, but the telescope must be revolved in a vertical plane until the intersection of the cross-hairs falls at a division upon the rod equal to the height of the cross-hairs above the ground at the transit station. If now the rod be held perpendicular to the line of sight, and the gradienter-screw turned through two revolutions, the intercept upon the rod will be $\frac{1}{100}$ of the required distance. With the gradienter, as with the stadia, it is more convenient to hold the rod vertical and apply the necessary correction to the rod reading.

Set up the transit over a point at one end of the line and level carefully. Measure the height of the cross-hairs above the ground. Now loosen the clamp of the tangent-screw attached to the vertical arc or circle, and revolve the telescope in a vertical plane until the intersection of the cross-hairs falls upon a point C (Fig. 98) upon the rod held at D, such that CD = AB. Read and note the vertical angle after clamping the gradienter-screw with both scales set to read zero. This angle will be ϕ (Fig. 98). Now turn the gradienter-screw through two revolutions, and note the reading ED upon the rod; the difference between this and CD (= AB) will give EC, which call S'; let FC, the perpendicular intercept upon the rod, be called S; the distance AC, parallel to the slope, H'; and let the horizontal distance AG be denoted by H. Then from the figure,

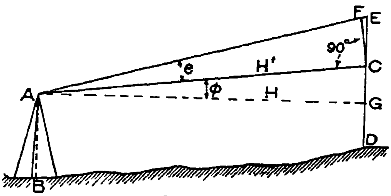


Fig. 98.

$$H' = 100\,S.$$

Now, from the right triangle EAG, the angle at E $= 90° - (\theta + \phi)$; and from the right triangle FAC, the angle at F $= 90° - \theta$. Therefore, in the triangle CFE, the angle at F $= 180° - (90° - \theta) = 90° + \theta$.

Therefore,

$$S : S' :: \sin\,[90° - (\theta + \phi)] : \sin\,(90 + \theta);$$

$$\text{or } S : S' :: \cos\,(\theta + \phi) : \cos\theta.$$

Hence $S = S' \dfrac{\cos(\theta + \phi)}{\cos\theta}$

$$= S' \frac{\cos\theta \cos\phi - \sin\theta \sin\phi}{\cos\theta} = S' (\cos\phi - \sin\phi \tan\theta);$$

but $\tan\theta = \dfrac{1}{100}$, and therefore $S = S' (\cos\phi - \sin\phi \dfrac{1}{100})$.

Therefore H′, the distance along the slope,

$$= S' (100 \cos\phi - \sin\phi);$$

and H, the horizontal distance,

$$= H' \cos\phi = S' (100 \cos^2\phi - \cos\phi \sin\phi)$$

$$= S' (100 \cos^2\phi - \tfrac{1}{2} \sin 2\,\phi)$$

$$= 100\,S' - S' (100 \sin^2\phi + \tfrac{1}{2} \sin 2\,\phi).$$

It may be well to note that the lower reading of the rod need not necessarily be such as to make CD = AB, but only as a matter of convenience.

EXAMPLE FOR PRACTICE.

Upper rod reading = 7 49
Lower rod reading = 4.67
Vertical angle of lower rod reading = 15° 35′.

Required to find the distance parallel to the slope between B and D and the horizontal distance AG.

Let it now be required to find the difference of elevation between B and D = CG.

Evidently $CG = H \tan\phi = S' (100 \cos^2\phi \tan\phi - \cos\phi \sin\phi \tan\phi)$.

$$= S' (100 \sin\phi \cos\phi - \sin^2\phi)$$

$$= S' (100\,\tfrac{1}{2} \sin 2\,\phi - \sin^2\phi).$$

In the last example, determine the difference of level of B and D.

It must not be forgotten of course, that the gradienter used in this way cannot give results so accurately as the spirit level:

but nevertheless, for rapid work, the results will be sufficiently correct.

If the student possesses a set of stadia reduction tables, the values of $\sin^2 \phi$ and $\frac{1}{2} \sin 2 \phi$ can be taken out at once and much labor saved.

To Lay out a Meridian with the Transit. *By means of the North Star at Upper or Lower Culmination.* Twice in 24 hours (more exactly, 23 hours 56 minutes) the north star "culminates"; that is to say, it attains to its maximum distance from the pole, above or below it. At the moment of culmination, the star is upon the meridian and if, therefore, a line be ranged out upon the ground in the same vertical plane, it will define a meridian.

Set up the transit over a peg, in an open space, giving an unobstructed view of a line about 400 or 500 feet long. Level the instrument carefully (it should be in perfect adjustment), and, a few minutes before the time of culmination, as given in the table, focus the intersection of the cross-hairs upon the star; clamp the plates, the vertical axis, and the horizontal axis of the telescope. Now by means of the tangent-screws attached to the vertical axis and to the vertical circle, move the telescope in azimuth and altitude, keeping the cross-hairs fixed upon the star. After a time it will be found that the position of the star no longer changes in altitude; it is then upon the meridian. Now clamp the vertical axis, plunge the telescope, and carefully center a stake 400 or 500 feet from the instrument; the line connecting the two stakes, will define the true meridian.

The whole operation may be repeated several nights in succession, and the mean of all the results taken.

By Means of the North Star at Eastern or Western Elongation. Twice in 24 hours, the north star attains to its maximum distance east or west of the pole, called its eastern or western "elongation." If a line be ranged out upon the ground in the direction of the star—at, say, the time of eastern elongation, and again at the time of western elongation—and if the angle between these two lines be bisected by a third line, this last line will evidently be a true north and south line.

Otherwise. Having laid out a line upon the ground in the a table the azimuth (or bearing) of the star at such time, and upon the horizontal plate of the transit set off this angle to the east and range out a line—which will therefore be a true north and south line. If the position of the star is taken at eastern elongation, the azimuth must be turned off to the west.

Set up the transit over a peg a few minutes before the star attains its maximum elongation, as given by the table. Level, and fix the line of collimation upon the star, following its movement as described under the previous method. After a time, it will be found that the movement of the star in azimuth ceases; the star has then attained its maximum elongation. Now clamp the vertical axis of the instrument, plunge the telescope, and center a stake in the proper direction. Now take from the table the proper azimuth, revolve the upper plate through the given angle in the proper direction, and range out a line upon the ground for the true meridian.

In order to determine the azimuth of the north star at eastern or western elongation, it is necessary to know the latitude of the place of observation.

Definitions. The **altitude** of a star is the vertical angle at the instrument included between the plane of the horizon and the line from the instrument to the star, as given by the line of collimation.

The **latitude** of a place is equal to the altitude of the pole.

If, therefore, we have any method of determining the altitude of the pole, the latitude of the observer is known at once.

The altitude of the pole may be determined by observing the altitude of the north star, first at its upper culmination and again at its lower culmination. The mean of these observations, corrected for refraction, will give the altitude of the pole and therefore the latitude of the observer. See tables of refraction of Polaris.

Set up the transit and level it, and proceed in the same manner as described under the first method for laying out a true meridian. When the star has reached its maximum distance above or below the pole, as indicated by the line of collimation moving in a horizontal plane, clamp the horizontal axis of the telescope and read the angle upon the vertical circle. The result will be the altitude of the star, say at upper culmination. Repeat the operation at lower culmination. Now, if A represents the altitude at upper culmination, and A_1 the altitude at lower culmination; d the refraction at upper culmination, and d_1 the refraction at lower culmination, then A_p, the altitude of the pole (= latitude of the place), will be given by the following:

$$A_p = \tfrac{1}{2}(A + A_1 - d - d_1)$$

It will be well to repeat these observations and take the mean of the results as the probable altitude of the pole.

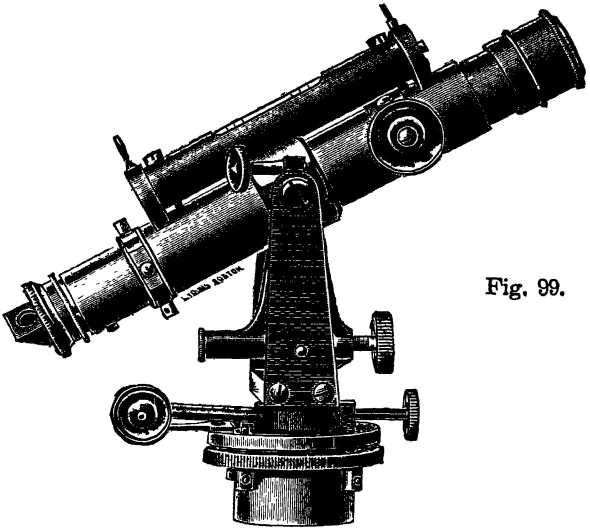


Fig. 99.

THE SOLAR TRANSIT.

The solar transit is an ordinary engineer's transit fitted with a solar attachment. Of the many forms of solars in use, that invented by G. N. Saegmuller, Washington, D. C., seems to be the favorite. In its latest form it is shown in Fig. 99, and consists of a telescope and level attached to the telescope of the transit (see Fig. 100) in such a manner as to be free to revolve in two directions at right angles to each other. When the transit telescope is horizontal and the bubble of the solar in the center of its tube, the auxiliary telescope with its bubble revolves in horizontal and ver-

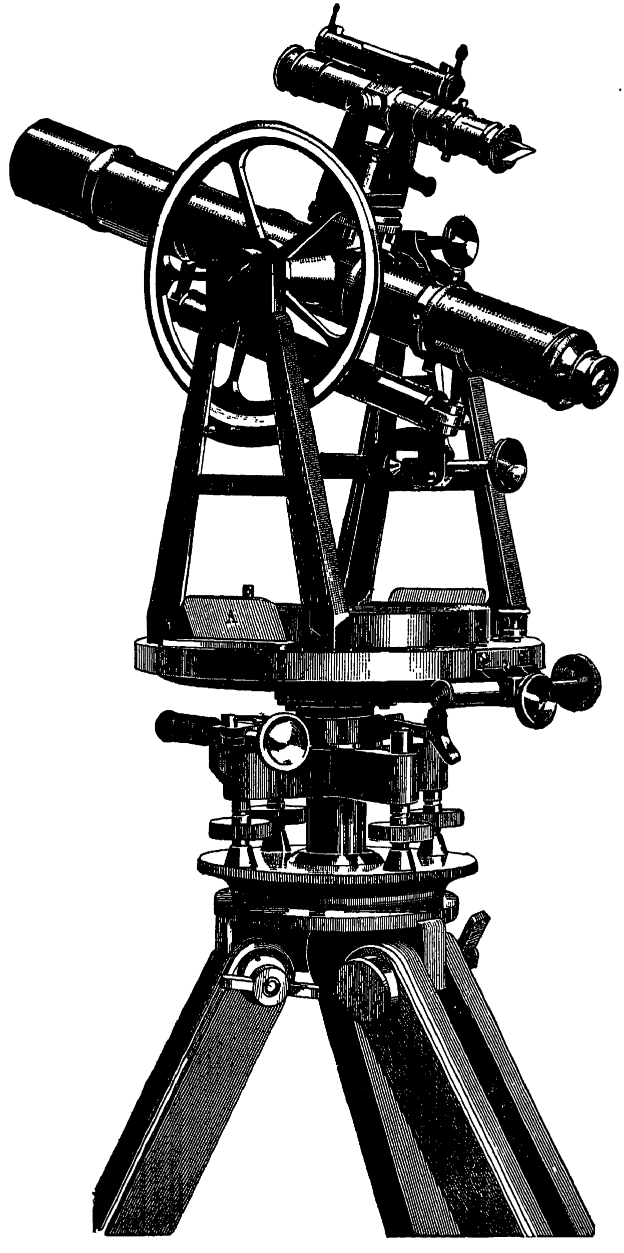


Fig. 100.

If now the line of collimation of the transit be brought into the meridian, the telescope pointing to the south, then, if we lay off upon the vertical circle, upward, the co-latitude of the place, the polar axis of the solar will be parallel to the axis of the earth. If now the two lines of sight are parallel and the solar telescope is revolved upon its polar axis, it is evident that its line of sight will describe a plane parallel to the plane of the equator. If now the transit telescope be still maintained parallel to the equator, if we turn the solar telescope upon its horizontal axis until the angle between the two lines of collimation equals the declination of the sun, then when the solar telescope is revolved upon its polar axis, its line of collimation will follow the path of the sun for the given day, provided there be no change in the sun's declination. If therefore the solar telescope is revolved until the image of the sun is brought between a pair of horizontal and vertical wires, provided in the telescope for that purpose, at that instant the line of sight of the transit telescope is in the meridian.

The horizontal axis of the solar telescope and the polar axis of the solar are provided with clamps and tangent-screws by means of which careful adjustments may be made. Two pointers are attached to the solar telescope, so adjusted that when the shadow of the one is thrown upon the other, the sun will appear in the field of view. There are also provided colored glass shades to the eyepiece to protect the eye when observing upon the sun. The objective and the cross-hairs are focused in the usual way.

Adjustments of the Solar Transit. It is assumed in what follows that the transit is in perfect adjustment, particularly the plate levels, the horizontal axis of the telescope, and the zero of the vertical circle.

1. *To adjust the Polar Axis.* The polar axis should be vertical when the line of collimation and the horizontal axis of the telescope are horizontal. To make this adjustment, level the transit by means of the plate levels. If the telescope is not fitted with a level, make the vernier of the vertical circle read zero. Now bring the bubble of the solar to the center of its tube and clamp the horizontal axis. Loosen the clamp of the polar axis, and turn the solar upon its polar axis through 180°. If the bubble remains

bubble runs toward one end of the tube, correct one-half of the error by revolving the solar telescope upon its horizontal axis and the other half by means of the capstan-headed screws at the base of the solar.

If the telescope of the transit is fitted with a level, it will be better to test the verticality of the vertical axis by means of it, since it is longer and more sensitive than the bubbles upon the plate. To do this, revolve the telescope upon its vertical axis until it is directly over a pair of diagonally opposite plate screws, and bring the bubble to the center by means of the tangent-screw attached to the horizontal axis of the telescope. Now revolve the telescope upon its vertical axis through 180°, and note if the bubble runs to one end; if it does correct one-half the error by the parallel plate-screw and the other half by the tangent-screw of the horizontal axis, and repeat this test and correction until the bubble remains in the center in all positions.

2. *To Adjust the Cross-Hairs of the Solar.* The line of collimation of the solar telescope should be parallel to the line of collimation of the transit telescope. The first adjustment having been made, first bring the telescope into the same vertical plane by centering a stake by the transit telescope and clamping the vertical axis. Now turn the telescope of the solar upon the polar axis until the intersection of the cross-hairs covers the same point upon the stake, and clamp the polar axis. Now level both telescopes by bringing the bubbles to the center, and measure the distance between the axes of the two telescopes; draw at this distance two black parallel lines upon a piece of white paper. Tack up the paper against a wall, post, or other convenient object, adjusting it in position so that one black line is covered by the horizontal cross-hair of the transit telescope; notice if the other black line is covered by the horizontal cross-hair of the solar; if so, the adjustment is completed; otherwise, move the diaphragm carrying the cross-hairs of the solar, until the second black line is covered. Adjusting the cross-hair diaphragm may displace the solar telescope vertically, so that the bubble should again be brought to the center of the tube, and the adjustment tested and repeated until the two lines of collimation are parallel, when the two bubbles are simul-

The Use of the Solar Transit. An ob
solar transit involves four quantities as follows:

1. The time of day, that is to say, the hour-angle of the sun.
2. The declination of the sun.
3. The latitude of the place of observation.
4. The direction of the meridian.

Any three of these quantities being known, the fourth may be determined by direct observation. The principal use of the solar transit is to determine a true meridian when the other three quantities are known.

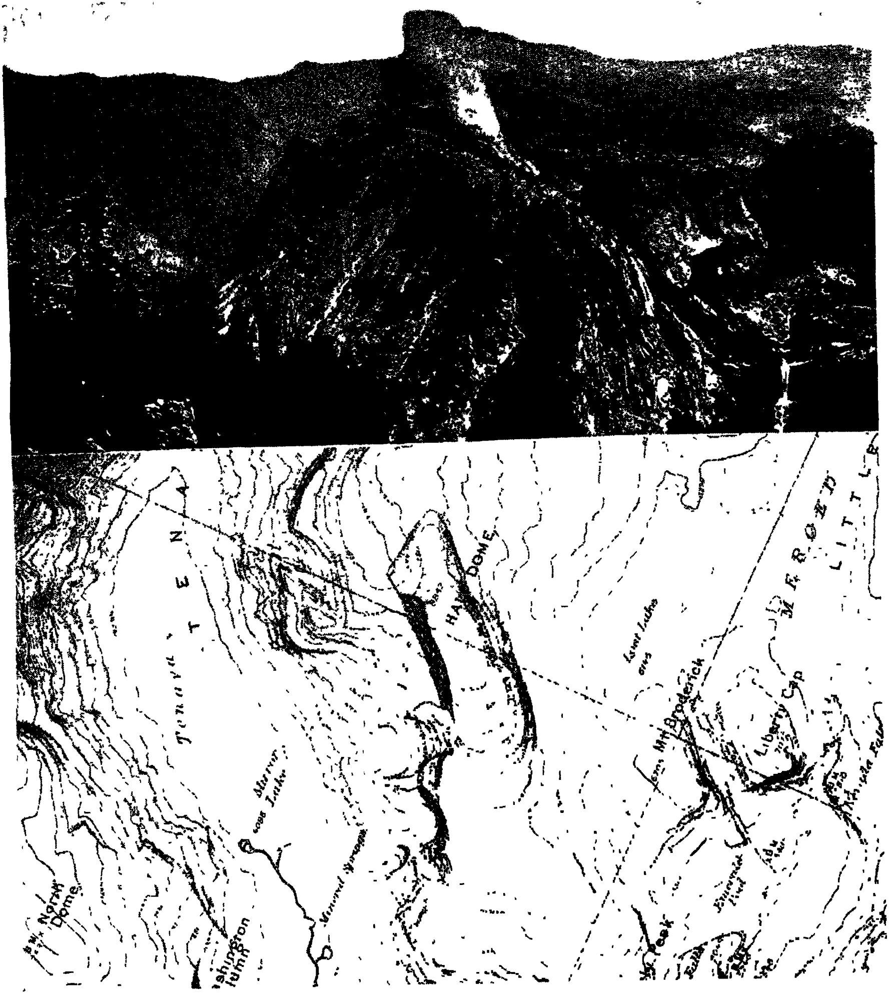
To Lay Out a True Meridian. Set up the transit over a stake; level the instrument carefully; and bring the lines of collimation of the telescopes, into the same vertical plane by the method previously described. Take the declination of the sun as given in the *Nautical Almanac* for the given day, and correct it for refraction and hourly change. Revolve the *transit telescope* upon its horizontal axis so that the vertical circle will record this corrected declination, turning it down if the declination is north, and elevating it if the declination is south. Now, without disturbing the position of the transit telescope, bring the solar telescope to a horizontal position by means of the attached level. It is evident that the angles between the lines of collimation will equal the corrected declination of the sun, and the inclination of the solar telescope to its polar axis will be equal to the polar distance of the sun.

Next, without disturbing the *relative* positions of the two telescopes, set the vernier of the transit telescope to the co-latitude of the place, and clamp the horizontal axis. It is evident that the transit telescope is parallel to the equator, and that the solar telescope is in a position to describe the path of the sun when the line of collimation of the transit *is in the true meridian*; and unless the line of collimation is in the true meridian, the sun cannot be brought between the cross-hairs of the solar telescope. Therefore unclamp the vertical axis of the transit and the polar axis of the solar, and, maintaining the *relative* positions of the telescopes revolve the transit upon its vertical axis, and the solar upon its polar axis, until the sun is brought between the cross-hairs of the solar telescope. Now clamp the vertical axis of the transit and

The solar apparatus should not be used between 11 A. M. and 1 P. M. if the best results are desired. From 7 to 10 A. M. and from 2 to 5 P. M. in the summer will give the best results. The greater the hour-angle of the sun, the better the observation will be so far as instrumental errors are concerned. However, if the sun is too close to the horizon, the uncertainties in regard to refraction will cause unknown errors of considerable magnitude.

Observation for Time. If the two telescopes—being in position, one in the meridian and the other pointing to the sun—are now revolved upon their *horizontal* axes (the vertical remaining undisturbed) until each is level, the angle upon the horizontal plate between their directions, as found by sighting on a distant object, will give the time from apparent noon, reliable to within a few seconds.

To Determine the Latitude. Level the transit carefully, and point the telescope toward the south, setting off the declination of the sun upon the vertical circle, elevating the object end if the declination is south, and depressing it if the declination is north. Bring the telescope of the solar into the same vertical plane with the transit telescope by the method previously described, level it carefully, and clamp it. The angle between the lines of collimation will then equal the declination of the sun. With the solar telescope, observe the sun a few minutes before its culmination, by moving the *transit telescope in altitude and azimuth* until the image of the sun is brought between the cross-hairs of the solar, keeping it there by means of the tangent-screws until the sun ceases to rise. Then take the reading of the vertical circle, correct for refraction due to altitude by the table below, subtract the result from 90°, and the remainder is the latitude sought.



PHOTOGRAPHIC VIEW AND CONTOUR MAP IN YOSEMITE VALLEY, CALIFORNIA

These two reproductions of the same area give a good idea of the two methods of showing topographical features

Courtesy of the United States Geological Survey, Washington, D C

Mean Refraction at Various Altitudes.*

Barometer, 30 inches. Fahrenheit Thermometer, 50°.

Altitude.	Refraction.	Altitude.	Refraction.
10°	5′ 19″	20°	2′ 39″
11	4 51	25	2 04
12	4 27	30	1 41
13	4 07	35	1 23
14	3 49	40	1 09
15	3 34	45	58
16	3 20	50	49
17	3 08	60	34
18	2 57	70	21
19	2 48	80	10

Preparation of the Declination Settings for a Day's Work. The solar ephemeris gives the declination of the sun for the given day, for Greenwich mean noon. Since all points in America are west of Greenwich, by 4, 5, 6, 7, or 8 hours, the declination found in the ephemeris is the declination at the given place at 8, 7, 6, 5, or 4 o'clock A. M. of the same date, according as the place lies in the "Colonial," "Eastern," "Central," "Mountain," or "Pacific" time belts respectively.

The columns headed "Refraction Corrections" (see table) give the correction to be made to the declination, for refraction for any point whose latitude is 40°. If the latitude is more or less than 40°, these corrections are to be multiplied by the corresponding coefficient given in the table of "Latitude Coefficients" (page 148). Thus the refraction corrections in latitude 30° are 65 one-hundredths, and those of 50 142 one-hundredths of the corresponding ones in latitude 40°. There is a slight error in the use of these latitude coefficients, but the maximum error will not amount to over 15 seconds, except when the sun is very near the horizon, and then any refraction becomes very uncertain. All refraction tables are made out for the mean (or average) refraction whereas the actual refraction at any particular time and place may be not more than one-half or as much as twice the mean refraction, with small altitudes. The errors made in the use of these latitude coefficients are therefore very small compared with the errors resulting from the use of the mean, rather than unknown actual, refraction which affects any given observation.

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*This table, as well as those following, is taken from the catalogue of

Latitude Coefficients.

LAT.	COEFF.	LAT.	COEFF.	LAT.	COEFF.	LAT.	COEFF.
15°	.30	27°	.56	39°	.96	51°	1.47
16	.32	28	.59	40	1.00	52	1.53
17	.34	29	.62	41	1.04	53	1.58
18	.36	30	.65	42	1.08	54	1.64
19	.38	31	.68	43	1.12	55	1.70
20	.40	32	.71	44	1.16	56	1.76
21	.42	33	.75	45	1.20	57	1.82
22	.44	34	.78	46	1.24	58	1.88
23	.46	35	.82	47	1.29	59	1.94
24	.48	36	.85	48	1.33	60	2.00
25	.50	37	.89	49	1.38		
26	.53	38	.92	50	1.42		

If the date of observation be between June 20 and September 20, the declination is positive and the hourly change negative; while if it be between December 20 and March 20, the declination is negative and the hourly change positive. The refraction correction is always positive; that is, it always increases numerically the north declination, and diminishes numerically the south declination. The hourly refraction corrections given in the ephemeris are exact each for the middle day of the five-day period, corresponding to that of hourly corrections. For the extreme days of any such period, an interpolation can be made between the adjacent hourly corrections, if desired.

By using standard time instead of local time, a slight error is made, but the maximum value of this error is found at those points when the standard time differs from the local time by one-half hour, and in the spring and fall when the declination is changing rapidly. The greatest error then, is less than 30 seconds, and this is smaller than can be set off on the vertical circle or declination arc. Even this error can be avoided by using the true difference of time from Greenwich in place of standard meridian time.

EXAMPLES FOR PRACTICE.

(1) Let it be required to prepare a table of declination for June 10, 1904, for a point whose latitude is 40° 20′, and which lies in the "Central Time" belt.

Since the time is 6 hours earlier than that at Greenwich, the declination given in the ephemeris is the declination at the given place at 6 A. M. of the same date. This is found to be 23° 0′ 18″. To this must be added the hourly change which is also plus and equal to 11.67″. The latitude coefficient is 1.013. The following table may now be made out.

HOUR	DECLINATION	REF COR	SETTING	HOUR	DECLINATION	REF. COR	SETTING
7	+ 23° 0′ 30″	+ 1′ 10″	23° 1′ 40″	1	23° 1′ 41″	18″	23° 1′ 59″
8	+ 23° 0′ 41″	+ 44″	23° 1′ 25″	2	23° 1′ 52″	22″	23° 2′ 14″
9	+ 23° 0′ 53″	+ 29″	23° 1′ 22″	3	23° 2′ 04″	29″	23° 2′ 33″
10	+ 23° 1′ 5″	+ 22″	23° 1′ 27″	4	23° 2′ 16″	44″	23° 3′ 0″
11	+ 23° 1′ 17″	+ 18″	23° 1′ 35″	5	23° 2′ 28″	1′ 10″	23° 3′ 38″

PROBLEMS INVOLVING USE OF TRANSIT.

Perpendiculars and Parallels. *To erect a perpendicular to a line at a given point of the line.* Set up the transit over the given point, and with the verniers set to 0°, direct the line of sight along the given line. Clamp the lower motion, unclamp the upper motion, and turn off an angle of 90° in the proper direction for the required line.

To erect a perpendicular to an inaccessible line at a given point of the line. Let AB, Fig. 101, be the given inaccessible line, and A the point of the line at which it is proposed to erect the perpendicular AD. Select some point H from which can be distinctly seen the points A and B of the inaccessible line. Set up the transit at the point H, and measure the angle AHB. Also from the point H run out and measure a line of any convenient length, and in such a direction that the points A and B can be seen from its extremity, as E. Now measure the angles AHE and BHE. Now set up the transit at E, and measure the angles BEH, BEA, and AEH. In the triangle AHE, we know from measurement the length of the side HE, as also the angles AHE and AEH, from which may be calculated the length of the side AH, which is also one side of the triangle AHB. From

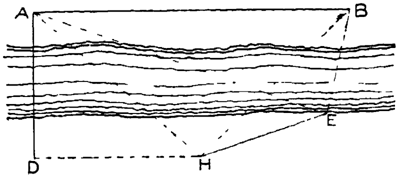


Fig. 101.

Refraction Correction.

Latitude, 40°.

January.

		′	″
1 2 3	1h. 2 3 4	1 2 3 6	58 16 04 23
4 5 6 7 8	1 2 3 4	1 2 2 6	54 11 59 01
9 10 11 12 13	1 2 3 4	1 2 2 5	51 07 51 40
14 15 16 17 18	1 2 3 4	1 2 2 5	46 01 40 00
19 20 21 22 23	1 2 3 4	1 1 2 4	42 56 31 35
24 25 26 27 28	1 2 3 4	1 1 2 4	37 58 22 07
29 30 31	1 2 3 4h.	1 1 2 3	32 44 13 41

February.

		′	″
1 2			
3 4 5 6 7	1h. 2 3 4	1 1 2 3	26 37 04 21
8 9 10 11 12	1 2 3 4	1 1 1 3	21 31 56 04
13 14 15 16 17	1 2 3 4 5	1 1 1 2 8	16 25 48 47 39
18 19 20 21 22	1 1 3 4 5	1 1 1 2 6	12 20 40 31 49
23 24 25 26 27	1 2 3 4 5h.	1 1 1 2 5	07 15 33 18 28
28			

March.

		′	″
1 2 3 4	1h. 2 3 4 5	1 1 1 2 4	03 10 27 06 39
5 6 7 8 9	1 2 3 4 5	0 1 1 1 4	59 06 21 56 04
10 11 12 13 14	1 2 3 4 5	0 1 1 1 3	55 02 15 47 34
15 16 17 18 19	1 2 3 4 5	0 0 1 1 3	52 58 10 39 08
20 21 22 23 24	1 2 3 4 5	0 0 1 1 2	48 54 05 32 51
25 26 27 28 29	1 2 3 4 5	0 0 1 1 2	45 50 01 25 34
30 31	1 2h.	0 0	42 47

April.

		′	″
1 2 3	3h. 4 5	0 1 2	57 19 18
4 5 6 7 8	1 2 3 4 5	0 0 0 1 2	39 44 54 14 08
9 10 11 12 13	1 2 3 4 5	0 0 0 1 1	36 41 51 10 58
14 15 16 17 18	1 2 3 4 5	0 0 0 1 1	34 38 48 06 49
19 20 21 22 23	1 2 3 4 5	0 0 0 1 1	32 36 45. 02 42
24 25 26 27 28	1 2 3 4 5	0 0 0 0 1	30 34 42 58 36
29 30	1 2h.	0 0	28 32

May.

		′	″
1 2 3	1h. 2 3 4 5	0 0 0 0 1	28 32 39 55 30
4 5 6 7 8	1 2 3 4 5	0 0 0 0 1	26 30 37 53 26
9 10 11 12 13	1 2 3 4 5	0 0 0 0 1	25 29 36 51 22
14 15 16 17 18	1 2 3 4 5	0 0 0 0 1	23 27 34 49 18
19 20 21 22 23	1 2 3 4 5	0 0 0 0 1	22 26 33 47 15
24 25 26 27 28	1 2 3 4 5	0 0 0 0 1	21 25 32 46 13
29 30 31	1 2 4 4 5h.	0 0 0 0 1	20 24 31 44 11

June.

		′	″
1, 2	5h.	1	11
3 4 5 6 7	1 2 3 4 5	0 0 0 0 1	19 23 30 43 10
8 9 10 11 12	1 2 3 4 5	0 0 0 0 1	18 22 29 43 09
13 14 15 16 17	1 2 3 4 5	0 0 0 0 1	18 22 29 42 08
18 19 20 21 22	1 2 3 4 5	0 0 0 0 1	18 22 28 42 08
23 24 25 26 27	1 2 3 4 5	0 0 0 0 1	18 22 29 42 08
28 29 30	1 2 3 4h.	0 0 0 0	18 22 29 43

July.

		′	″
1 2	5h.	1	09
3 4 5 6 7	1 2 3 4 5	0 0 0 0 1	19 23 30 43 10
8 9 10 11 12	1 2 3 4 5	0 0 0 0 1	20 24 31 44 11
13 14 15 16 17	1 2 3 4 5	0 0 0 0 1	21 25 32 46 13
18 19 20 21 22	1 2 3 4 5	0 0 0 0 1	22 26 33 47 15
23 24 25 26 27	1 2 3 4 5	0 0 0 0 1	23 27 34 49 18
28 29 30 31	1 2 3 4 5h.	0 0 0 0 1	25 29 36 51 22

August.

		′	″
1			
2 3 4 5 6	1h. 2 3 4 5	0 0 0 0 1	26 30 37 53 26
7 8 9 10 11	1 2 3 4 5	0 0 0 0 1	28 32 39 55 30
12 13 14 15 16	1 2 3 4 5	0 0 0 0 1	30 34 42 58 36
17 18 19 20 21	1 2 3 4 5	0 0 0 1 1	32 36 45 02 42
22 23 24 25 26	1 2 3 4 5	0 0 0 1 1	34 38 48 06 49
27 28 29 30 31	1 2 3 4 5h.	0 0 0 1 1	36 41 51 10 58

September.

		′	″
1 2 3 4 5	1h. 2 3 4 5	0 0 0 1 2	39 44 54 14 08
6 7 8 9 10	1 2 3 4 5	0 0 0 1 2	42 47 57 19 18
11 12 13 14 15	1 2 3 4 5	0 0 1 1 2	45 50 01 25 34
16 17 18 19 20	1 2 3 4 5	0 0 1 1 2	48 54 05 32 51
21 22 23 24 25	1 2 3 4 5	0 0 1 1 3	52 58 10 39 08
26 27 28 29 30	1 2 3 4 5h.	0 1 1 1 3	55 02 15 47 34

October.

		′	″
1 2 3 4 5	1h. 2 3 4 5	0 1 1 1 4	59 06 21 56 01
6 7 8 9 10	1 2 3 4 5	1 1 1 2 4	03 10 27 06 39
11 12 13 14 15	1 2 3 4 5	1 1 1 2 5	07 15 33 18 39
16 17 18 19 20	1 2 3 4 5	1 1 1 2 6	12 20 40 31 29
21 22 23 24 25	1 2 3 4 5	1 1 1 2 8	16 25 48 47 39
26 27 28 29 30	1 2 3 4 5	1 1 1 3 11	21 31 56 04 01
31	1h.	1 1	26 37

November.

		′	″
1 2 3 4	2h. 3 4 5	3 13	21 57
5 6 7 8 9	1 2 3 4 5	1 1 2 3	32 44 13 41
10 11 12 13 14	1 2 3 4 5	1 1 2 4	37 50 22 07
15 16 17 18 19	1 2 3 4 5	1 1 2 4	42 56 31 35
20 21 22 23 24	1 2 3 4 5	1 2 2 4	46 01 40 59
25 26 27 28 29	1 2 3 4 5h	1 2 2 5	50 06 19 33
30			

December.

		′	″
1 2 3 4	1h. 2 3 4 5	1 2 2 6	54 11 59 01
5 6 7 8 9	1 2 3 4 5	1 2 3 6	58 16 04 23
10 11 12 13 14	1 2 3 4 5	2 2 3 6	00 19 09 38
15 16 17 18 19	1 2 3 4 5	2 2 3 6	01 20 11 47
20 21 22 23 24	1 2 3 4 5	2 2 3 6	01 20 11 49
25 26 27 28 29	1 2 3 4 5h.	2 2 3 6	00 19 09 43
30 31			

ment, as well as the angles BHE and BEH, from which we can calculate the length of the side BH, which is also one side of the triangle AHB. Therefore in the triangle AHB, we have the lengths of the two sides AH and BH by calculation; and the angle AHB by measurement. We can therefore calculate the angle HAB, which equals the angle AHD. Set up the transit at H, sight to A, and turn off the angle AHD (= HAB), measuring off HD of a length equal to AH cos AHD. Then AD will be the perpendicular required, and its length will equal AH sin AHD.

The calculation is as follows: In the triangle AHE, the angle HAE = 180° − (AHE + AEH), and therefore AH : HE : : sin AEH : sin HAE; or, $\mathrm{AH} = \mathrm{HE}\,\dfrac{\sin \mathrm{AEH}}{\sin \mathrm{HAE}}$.

In the triangle HEB, HBE = 180° − (BHE + BEH), and therefore HB : HE : : sin HEB : sin HBE;

or $\mathrm{HB} = \mathrm{HE}\,\dfrac{\sin \mathrm{HEB}}{\sin \mathrm{HBE}}$.

In the triangle AHB, the sum of the angles HAB and HBA = 180° − AHB. Let x represent the difference of the angles HAB and HBA. Then, from trigonometry,

$$\mathrm{AH} + \mathrm{HB} : \mathrm{AH} - \mathrm{HB} :: \tan \tfrac{1}{2}\,(\mathrm{ABA} + \mathrm{HAB}) : \tan \tfrac{1}{2}\,(\mathrm{ABH} - \mathrm{HAB});$$

$$\text{or, } \mathrm{AH} + \mathrm{HB} : \mathrm{AH} - \mathrm{HB} :: \tan \tfrac{1}{2}\,(180^\circ - \mathrm{AHB}) : \tan \tfrac{1}{2}\,x;$$

$$\text{or, } \mathrm{AH} + \mathrm{HB} : \mathrm{AH} - \mathrm{HB} :: \cot \frac{\mathrm{AHB}}{2} : \tan \tfrac{1}{2}\,x.$$

From this last proportion we find x, the difference of the two angles HAB and HBA. We then have the simultaneous equations:

$$\mathrm{HAB} + \mathrm{HBA} = c \text{ (say)}$$
$$\mathrm{HAB} - \mathrm{HBA} = d \text{ (say)}$$

Therefore $\mathrm{HAB} = \dfrac{c + d}{2}$; and $\mathrm{HBA} = \dfrac{c - d}{2}$.

EXAMPLE FOR PRACTICE

Given HE (Fig. 101) = 125 feet; AHE = 122°; AHB = 94°; BHE = 28°; BEH = 121 ; BEA = 80 ; AEH = 41 . It is required to find the angle AHD, the length of HD, and the length of AD

Ans. AHD = 50°32′25″; HD = 229.46 feet; AD = 161.39 feet

To let fall a perpendicular to a line from a given point. Let AB, Fig, 102, be the given line, and C the point. Set up the transit at some point A of the given line, and measure the angle BAC. Take the instrument to C, sight to A and turn off an angle ACB = 90° − BAC. The instrument will then sight in the direction of the required perpendicular CB.

To let fall a perpendicular to a line from an inaccessible point. Let BC, Fig. 103, be the given line and A the inaccessible point from which it is desired to let fall the perpendicular upon BC. Set up the instrument, as at B; and, after measuring the length of BC, measure the angle ABC. Take the instrument to C, and measure the angle ACB. Then in the triangle ABC,

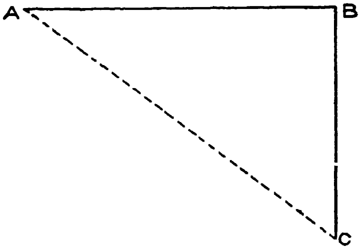


Fig. 102.

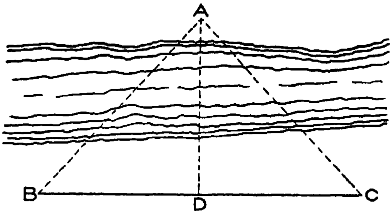


Fig. 103.

$$AB : BC :: \sin ACB : \sin (ACB + ABC);$$

or,
$$AB = BC \frac{\sin ACB}{\sin (ACB + ABC)};$$

$$BD = AB \cos ABC;$$

and,
$$BD = BC \frac{\tan ACB}{\tan ACB + \tan ABC}.$$

EXAMPLE FOR PRACTICE.

Given BC (Fig. 103) = 250 feet; ABC = 63°15′; ACB = 55 40′. Calculate the length of BD, and the length of AD.

$$\text{Ans.} \begin{cases} BD = 106.2 \text{ feet.} \\ AD = 210.7 \text{ feet.} \end{cases}$$

To let fall a perpendicular to an inaccessible line from a given point outside of the line. Let AB, Fig. 104, be the inaccessible line, and C the point from which it is desired to let fall the perpendicular to AB. Through C run out and measure a line of any convenient length, as CD, and measure the angles ACB

DCB, and DCA. Set up the instrument at D, and measure the angles ADC and BDC. In the triangle BDC, we have given two angles and the included side, from which can be calculated the length of the side CB, In the triangle ADC, we have given two angles and the included side, from which can be calculated the length of the side AC. Then, in the triangle ACB, we have the lengths of the sides AC and CB, and the included angle ACB, from which can be calculated the angle CAB. If, then, the instrument be set up at C, and an angle ACE be turned off equal to 90° − BAC, the line of sight will point in the direction of the required perpendicular, and the length of the perpendicular will be given by AC cos ACE.

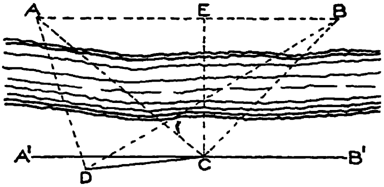
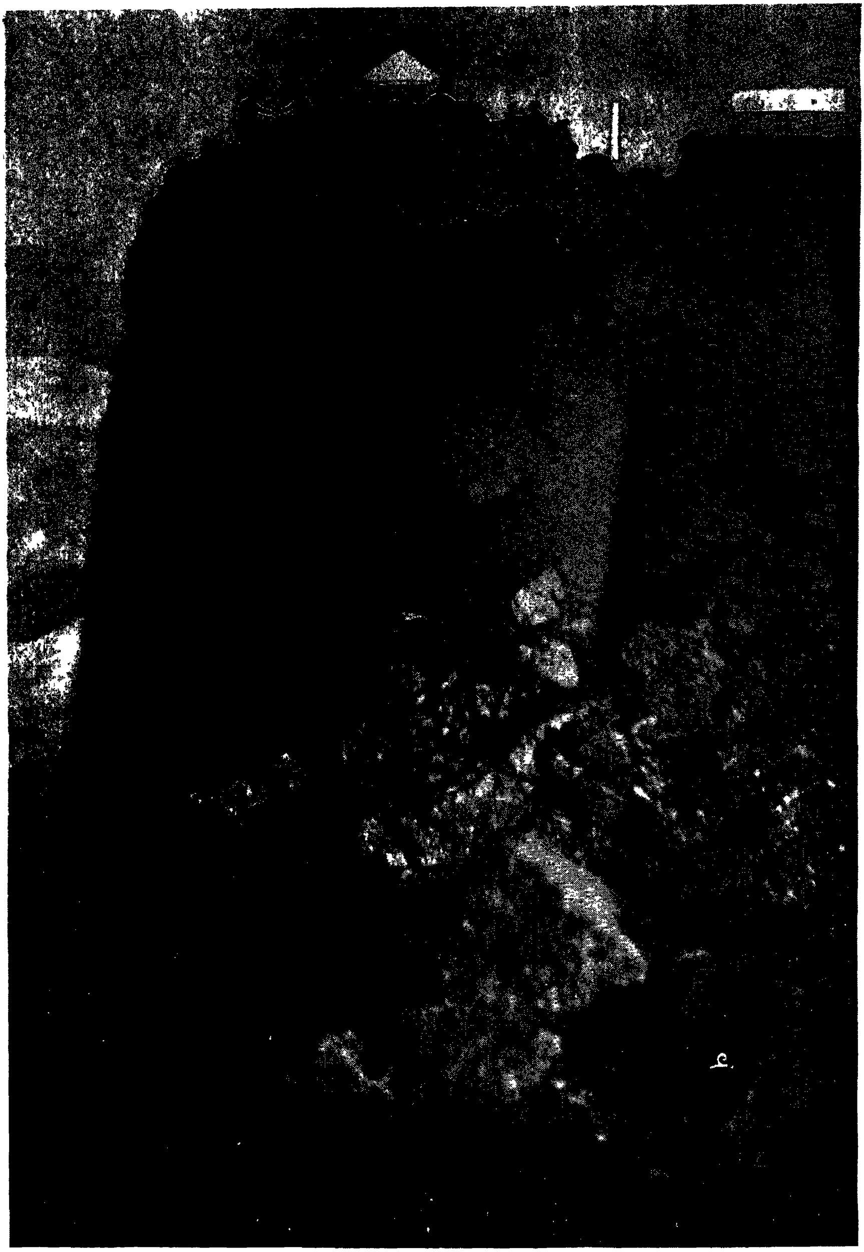


Fig. 104.

This same method will serve to *trace a line through a given point parallel to an inaccessible line.* For if, with the instrument at C, an angle ACA′ be turned off equal to CAB, the line A′ B′ will be parallel to AB.

Obstacles to Alignment. *By Perpendiculars:* When a tree, house, or other obstacle obstructing the line of sight (see Fig. 105) is encountered, set up the transit at the point B, turn off a right angle, and measure the length of the line BC. Erect a second perpendicular CD at C, and measure its length. At D erect a third perpendicular DE, making DE = BC. Then the fourth perpendicular EF will be in the direction of the required line. The distance from B to E will be given by CD. If perpendiculars cannot be conveniently set off, let BC and DE make any equal angle with the line AB, so that CD will be parallel to it.

By an Equilateral Triangle. At B turn off from the direction of AB produced, an angle of 60° in the direction of BC (see Fig. 106), and make BC any convenient length sufficient to clear the obstacle. Set up the instrument at C and turn off an angle of 60° from BC to CD and make CD of a length equal to BC Finally at D turn off a third angle of 60° from CD to DB, and the line DE will be in the direction of AB produced. The distance BD will equal BC or CD.



TRIANGULATION STATION ON MOUNT UNCOMPAHGRE. ELEVATION 14,300 FEET

Courtesy of U S Coast and Geodetic Survey, Washington, D C.

AC, and the correct line AB can be run out and measured in the proper direction.

By Latitudes and Departures. When a single line such as AC cannot be run so as to come opposite the given point B (Fig. 109), a series of zigzag lines (as AC, CD, DE, EF, and FB) can be run in any convenient direction, so as at last to arrive at the desired point B. Any one of these lines (as, for instance, AC) may be taken as a meridian to which all of the others may be referred, and their bearings therefrom deduced. Calculate the total latitudes and departures of these lines, as AX and BX; then the bearing of the required line BA with respect to AC will be given by Tan. BAC $= \frac{BX}{AX}$.

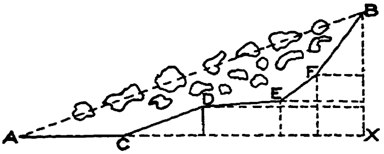


Fig. 109.

By Triangulation. When obstacles prevent the use of either of the preceding methods, if a point C can be found from which A and B are accessible (see Fig. 110), measure the distances CA and CB, and the angle ACB, from which can be calculated the length of the side AC and the angle CAB. Now measure the angle ACD to some point D beyond the obstacle; then, in the triangle ACD, we have two angles and the included side, from which may be calculated the length of the side CD. Measure the distance CD in the proper direction, set up the transit at D, and turn off an angle CDB equal to the supplement of ADC, for the direction of the required line.

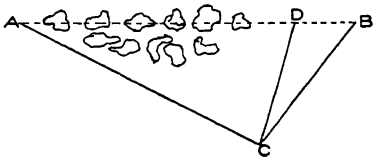


Fig 110

The distance from A to D may also be calculated from the triangle ACD, the stake at D given its proper number, and the line continued. If the distances CA and CB cannot be measured, it will be necessary to measure a base-line through C, from the extremities of which the angles to A and B can be measured and the required distances calculated as before.

The following problem, as illustrated in Fig. **111**, is of frequent occurrence in line surveys. The line AB of the survey having been brought up to one side of a stream, it is desired to continue the line of the survey across the stream to the point C, the latter point being visible from B and accessible. It is required to find the length of the line BC, that the stake at C may be given its proper number, and the survey continued from that point. With the transit at B, turn off the required angle to locate the point C, and drive a stake at that point. If possible, deflect from BC a right angle to some point E, and measure the length of BE. Take the transit to E, and measure the angle BEC. The distance BC is therefore:

$$BC = BE \tan BEC.$$

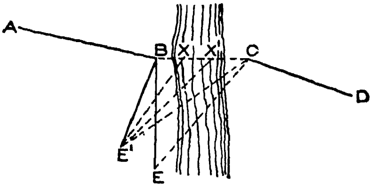


Fig. 111.

If it is not possible to turn off a right angle at B, then through B run a line (as BE') in any convenient direction, and measure its length; measure also the angles E'BC and BE'C. In the triangle CBE', there are then given two angles and the included side, from which the side BC can be calculated. Should it be necessary to take soundings at certain intervals (as, say, 50 or 100 feet across the stream), then in the triangle BE X there are given the distance BX, the distance E'X, and the angle XBE', from which can be calculated the angle BE'X. With the transit at E', turn off from BE' the angle BE'X. Now, starting a boat from the shore, direct it in line from B to C until it comes upon the line of sight of the transit from E' to X. At that point take soundings, and similarly for the point X', etc. If the point C is not visible from B, find some point, as E (see Fig. 112), from which B and C are visible, and measure the angle BEC and the distance EB. Find a second point, as F, from which E and C are visible, and measure the angles CEF and EFC and the distance EF. Then, in the triangle ECF, there are given two angles and the included side, from which can be calculated the distance EC. In the triangle BCE, then, there are given two sides and the included angle, and from these the third side BC and the angle EBC can be found. The stake C can now be numbered, and the bearing of BC deduced.

EXAMPLES FOR PRACTICE.

1. In Fig. 111, given BE′ = 210 feet; angle CBE′ = 110° 15′; angle BE′C = 34°20′; stake B numbered 8 + 54. It is required to find the number of the stake C.

2. (*a*) In Fig. 112, given EF = 250 feet; BE = 128 feet; angle EFC = 46° 40′; angle CEF = 103° 30′; angle BEC = 39° 10′. If the stake at B is numbered 12 + 20, it is required to find the number of the stake at C.

(*b*) If the bearing of the line AB is S 75°E, and the deflection angle of BE from AB is 104° to the *right*, find the bearing of BC.

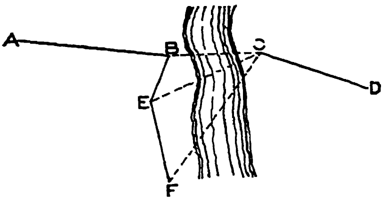


Fig. 112.

To Supply Omissions. Any two omissions in a closed survey—whether of the direction or of the length, or of both, of one or more lines of the survey—can always be supplied by the application of the principle of latitude and departures, although this method should be resorted to only in cases of absolute necessity, since any omission renders the checking of the field work impossible. In the following paragraphs, the methods outlined will apply equally whether the survey has been made with the transit or with the compass.

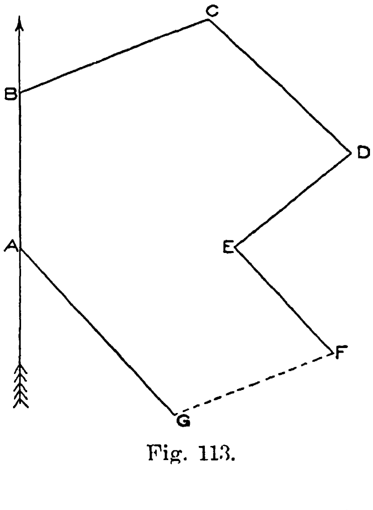


Fig. 113.

Case 1. *When the length and bearing of any one side are wanting.* In Fig. 113, let the dotted line FG represent the course whose length and bearing are wanting. Calculate the latitudes and departures of the remaining courses; and since in a closed survey the algebraic sum of the latitudes and departures should equal zero, therefore the difference of the latitudes will be the latitude of the missing line, and the difference of the longitudes will be the required longitude. The latitude and longitude of the line, form the sides of a right triangle, from which we have:

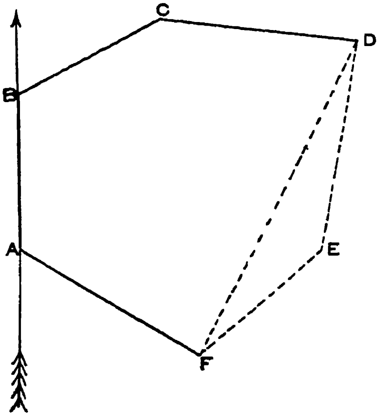


Fig. 114.

$$\text{Tangent of Bearing} = \frac{\text{Longitude}}{\text{Latitude}}.$$

The required length will be given by $L = \frac{\text{Latitude}}{\text{Cos Bearing}}$.

CASE 2. *When the length of one side and the bearing of another are wanting.*

(*a*) WHEN THE DEFICIENT SIDES ADJOIN EACH OTHER. In Fig. 114, let the bearing of DE, and the length of FE, be lacking. Draw DF. From the preceding proposition we can calculate the bearing and length of DF, as though DE and EF did not exist. Then, in the triangle DFE, we have given the lengths DF and DE and the angle DEF, from which can be calculated the angle FDE and the length EF.

(*b*) WHEN THE DEFICIENT SIDES ARE SEPARATED FROM EACH OTHER. In Fig. 115 let ABCDE FGA represent a seven-sided field, in which the length of CD, and the bearing of FG, are wanting. Draw DB′, B′A′, A′G′, of the same lengths, and parallel respectively to CB, BA, and AG. Connect G′ with GE and F. Then, in the figure DB′A′G′E, there are given the lengths and

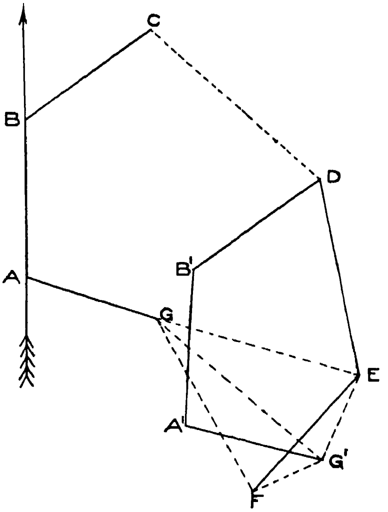


Fig. 115.

bearings of all of the courses but G′E. The length and bearing of the last course can be calculated by the principles of Case 1. Then, in the triangle EFG′, there are given the lengths and bearings of EF and EG′, from which can be calculated the length and bearing of FG′. Therefore, in the triangle GFG′, since GG′ is equal in length and parallel to CD, there are given the *lengths* of GF and FG′, and the *bearings* of GG′ and FG′, from which can be calculated the length of GG′ and the bearing of GF.

Case 3. *When the lengths of two sides are wanting.*

(*a*) When the deficient sides adjoin each other. In the seven-sided Fig. 116, let the lengths of DE and EF be wanting. Calculate the length and bearing of DF by the principles of Case 1. Then, in the triangle EDF, there are given the angles at D and F, and the length of DF, from which can be calculated the lengths of DE and EF.

(*b*) When the deficient sides are separated from each other. In Fig. 115, let the lengths of CD and GF be wanting. As before, having calculated the length and bearing of FG′, in the triangle FGG′, the angle at G can be calculated from the bearings of FG and GG′; the angle at G′ from the bearings of GG′ and FG′; and the angle at F from the bearings of FG and FG′. There are given then the three angles of the triangle, and the length of one side, from which can be calculated the lengths of the other sides.

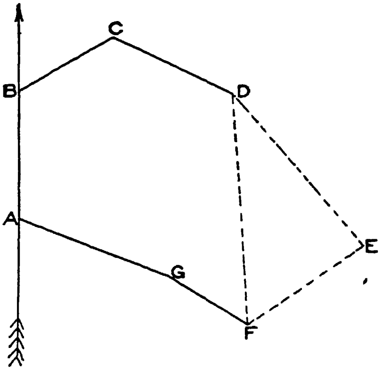


Fig. 116.

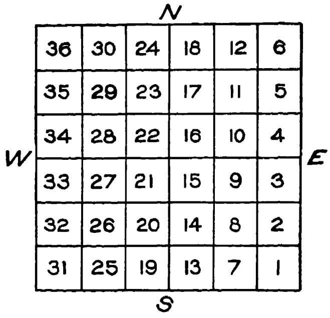
Case 4. *When the bearings of two sides are wanting.*

(*a*) When the deficient sides adjoin each other. In Fig. 116, find the length and bearing of DF as before. Then, in the triangle DEF, there are given the lengths of the three sides, from which can be calculated the required angles.

(*b*) When the deficient sides are separated from each other. In Fig. 115, let the bearings of CD and GF be wanting. Calculate the length and bearing of FG′ as before. Then, in the triangle FGG′, there are three sides known, from which can be calculated the three angles, and therefore the bearings can be deduced.

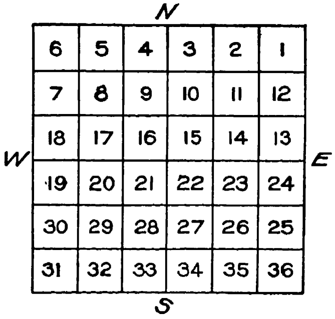
UNITED STATES PUBLIC LAND SURVEYS.

The first surveys of the public lands of the United States were carried out in Ohio, under an act of Congress approved May 20th, 1785. This act provided for townships 6 miles square, containing 36 sections of 1 mile square. The townships 6 miles square, were laid out in ranges, extending northward from the Ohio River, the townships being numbered from south to north, and the ranges from east to west. The territory embraced in these early surveys forms a part of the present state of Ohio and is known as "The Seven Ranges." The sections were numbered from 1 to 36 commencing with No. 1 in the *southeast* corner of the township, and running from *south* to *north* in each tier, to No. 36 in the *northwest* corner of the townships as shown in Fig. 117.



36	30	24	18	12	6
35	29	23	17	11	5
34	28	22	16	10	4
33	27	21	15	9	3
32	26	20	14	8	2
31	25	19	13	7	1

Fig. 117.



6	5	4	3	2	1
7	8	9	10	11	12
18	17	16	15	14	13
19	20	21	22	23	24
30	29	28	27	26	25
31	32	33	34	35	36

Fig. 118.

A subsequent act of Congress, approved May 18th, 1796, provided for the appointment of a surveyor general, and directed the survey of the lands northwest of the Ohio River, and above the mouth of the Kentucky River. This act provided that "the sections shall be numbered respectively, beginning with the number one in the northeast section, and proceeding west and east alternately, through the township, with progressive numbers till the thirty-sixth be completed." This method is shown in Fig. 118 and is still in use.

An act of Congress, approved Feb. 11th 1805, directs the subdivision of the public lands into quarter sections, and provides that all the corners marked in the public surveys shall be established as the proper corners of sections, or subdivisions of sections, which they were intended to designate, and that corners of half and quarter sections *not marked* shall be placed, as nearly as possible, "equidistant from those two corners which stand on the same line." This act further provides that "the boundary lines actually run and marked * * * shall be established as the proper boundary lines of the sections or subdivisions for which they were intended; and the length of such lines as returned by * * * the surveyors * * * shall be held and considered as the true length thereof". . . .

An act of Congress, approved April 24th, 1820, provides for the sale of public lands in half-quarter sections, and requires that "in every case of the division of a quarter section the line for the division thereof shall run north and south. An act of Congress, approved April 5th, 1832, directed the subdivision of the public lands into quarter-quarter-sections and that in every case of the division of a half-quarter section, the dividing line should run east and west; and that fractional sections should be subdivided under rules and regulations prescribed by the Secretary of the Treasury.

By an act of Congress, approved March 3rd, 1849, the Department of the Interior was created, and the act provided "That the Secretary of the Interior shall perform all the duties in relation to the General Land Office, of supervision and appeal now discharged by the Secretary of the Treasury. * * *" By this act the General Land Office was transferred to the Department of the Interior where it still remains.

The division of the public lands is effected by means of meridian lines and parallels of latitudes established six miles apart. The squares thus formed are called *Townships*, and contain 36 square miles, or 23,040 acres "as nearly as may be." All of the townships situated north or south of each other, form a *Range* and are named by their number east or west of the principal meridian. Thus, the first range west of the meridian would be designated as Range 1 West (R 1. W.). Each *tier* of townships is named by its number north or south of the base line, as Township 2 North

Existing laws further require that each township shall be divided into thirty-six sections, by two sets of parallel lines, one governed by true meridians and the other by parallels of latitude, the latter intersecting the former at right angles, at intervals of one mile; and each of these sections must contain, as nearly as possible, six hundred and forty acres. These requirements are evidently inconsistent because of the convergency of the meridians, and the discrepancies will be greater as the latitude increases.

In view of these facts, it was provided in section 3 of the act of Congress approved May 10th, 1800, that "in all cases where the exterior lines of the townships, thus to be subdivided into sections and half-sections, shall exceed, or shall not extend six miles, the excess or deficiency shall be specially noted, and added to or deducted from the western or northern ranges of sections or half-sections in such township, according as the error may be in running lines from east to west, or from south to north; the sections and half-sections bounded on the northern and western lines of such townships shall be sold as containing only the quantity expressed in the returns and plots, respectively, and all others as containing the complete legal quantity."

To harmonize these various requirements as fully as possible, the following methods have been adopted by the general land office.

Initial points are first established astronomically under special instructions, and from this initial point a "principal meridian" is laid out north and south. Through this initial point a "base line" is laid out as a parallel of latitude running east and west. On the principal meridian and base lines, the half-mile, mile and six-mile corners are permanently located, and in addition, the meander corners at the intersection of the line with all streams, lakes or bayous prescribed to be meandered. These lines may be run with solar instruments, but their correctness should be checked by observations with the transit upon Polaris at elongation.

Standard parallels, also called correction lines, are run east and west from the principal meridian at intervals of twenty-four miles north and south of the base line, and the law provides that "where standard parallels have been placed at intervals of thirty or thirty-six miles, regardless of existing instructions, and where gross irregularities require additional standard lines from which to initiate new, or upon which to close old surveys, an intermediate correction line should be established to which a *local* name may be given: and the same will be run, in all respects, like the regular standard parallels."

Guide meridians are extended *north* from the base line, or standard parallels, at intervals of twenty-four miles east and west of the principal meridian.

When conditions are such as to require the guide meridians to run *south* from a standard parallel or a correction line, they are initiated at properly established closing corners of the given parallel. That is to say, they are begun from the point on the parallel at which they would have met it if they had been run *north* from the next southern parallel. This point is obtained from computation, and is less than twenty-four miles from the next eastern or western meridian by the convergence of the meridians in twenty-four miles.

In case guide meridians have been improperly located too far apart, auxiliary meridians may be run from standard corners, and these may be designated by a local name.

The angular convergence of two meridians is given by the equation

$$\phi = m \sin L \tag{1}$$

where m is the angular difference in longitude of the meridians, and L is the mean latitude of the north and south length under consideration.

The linear convergence in a given length l is

$$c = l \sin \phi \tag{2}$$

The radius of a *parallel* at any latitude L is given by the equation

$$r = R \cos L \tag{3}$$

where R is the mean radius of curvature of the earth.

The distance between meridians is usually given in miles and this must be reduced to degrees. To do this it is first necessary to find the linear value of one degree of longitude at the mean latitude from the proportion.

$$1^\circ : 360^\circ :: x : 2\pi r \tag{4}$$

Equation (4) will give results sufficiently accurate, although in strict accuracy R should be the radius of curvature at the mean latitude.

For full details of public-land surveying, see "Manual of Surveying Instructions for the Survey of the Public Lands of the United States," issued by the Commissioner of the General Land Office. These "Instructions" are prepared for the direction of those engaged on the public land surveys, and new editions are issued from time to time.

Much of the foregoing in very condensed form is taken from the edition of 1894.

The following table gives the convergency both in angular units and linear units for township 6 miles square, between latitudes 30° and 70° north.

Let it be required to find from the table the linear convergence for a township situated in latitude 38° 29′ north.

Looking in the table opposite 39° we find the linear convergence.

For 39° = 58.8 links
For 38° = 56.8 links
Difference for 1° = 2.0 links
Difference for 1′ = 2.0 ÷ 60 = .0333 links
Difference for 29′ = .0333 × 29 = .97 links

Therefore total convergence for latitude 38° 29′ = 56.8 + 0.97 links = 57.77 links.

BASE MEASUREMENT.

It is not intended in what follows to go into the details of the measurement of a base for an extended system of triangulation, as that properly belongs to Geodetic Surveying. Some description of base measuring apparatus will be given, with illustrations of various devices, and special attention will be given to the use of the tape in the accurate measurement of lines such as occur in usual field operations of Plane Surveying.

Lat.	Convergency.			Lat.	Convergency		
	On the Parallel.	Angle.			On the Parallel.	Angle.	
Degrees.	Links.	Minutes.	Seconds.	Degrees.	Links.	Minutes.	Seconds.
30	41 9	3	0	50	86.4	6	12
31	43.6	3	7	51	89.6	6	25
32	45.4	3	15	52	92.8	6	39
33	47.2	3	23	53	96.2	6	54
34	49.1	3	30	54	99.8	7	9
35	50.9	3	38	55	103.5	7	25
36	52.7	3	46	56	107.5	7	42
37	54.7	3	55	57	111.6	8	0
38	56.8	4	04	58	116.0	8	19
39	58.8	4	13	59	120.6	8	38
40	60.9	4	22	60	125.5	8	59
41	63.1	4	31	61	130.8	9	22
42	65.4	4	41	62	136.3	9	46
43	67.7	4	51	63	142.2	10	11
44	70.1	5	1	64	148.6	10	38
45	72.6	5	12	65	155.0	11	8
46	75.2	5	23	66	162.8	11	39
47	77.8	5	23	67	170.7	12	13
48	80.6	5	46	68	179.3	12	51
49	83.5	5	59	69	188.7	13	31
				70	199 1	14	15

Much of what follows is from the excellent treatise on Topographic Surveying by Herbert M. Wilson.

A trigonometric survey is usually carried over a country where the direct measurement of distances is impracticable, and since the calculations of these distances proceeds from the *direct* measurement of the base-line, this base line should be so located as to permit of its length being determined with any degree of accuracy consistent with the nature of the work involved.

To attain the desired results, the site should be reasonably level and afford room for a base of proper length so that its ends primary triangulation giving the best-conditioned figures possible. Other things being equal, that site is best that includes solid ground; both for permanency of monuments and facility and accuracy of measurement.

Base Apparatus. In early days, base-lines were measured by means of wooden rods, varnished and tipped with metal. The rods were supported in trestles, the contacts between the ends being made with great care. Later, *compensated rods* were employed, as for instance the Contact-Slide Apparatus of the U. S. Coast Survey and the Repsold primary base bars of the U. S. Lake Survey, see Fig. 119, resulting in greater accuracy in the measurement of base lines. The use of the *iced bar* (see Fig. 120) by the U. S. Coast Survey, represents the highest development of base-measuring apparatus.

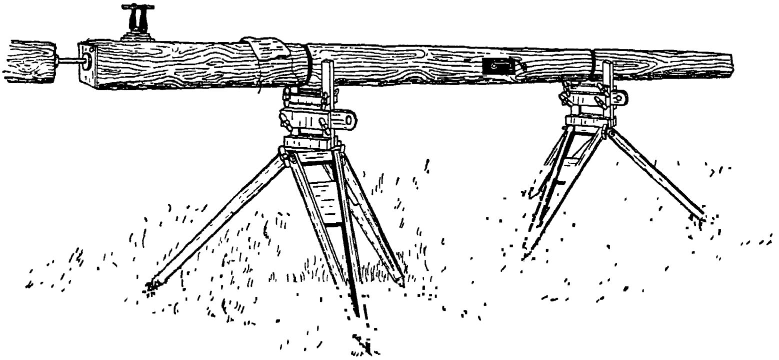


Fig. 119.

Within recent years the *steel tape* has become popular as the accuracy attainable with *its* use has become more fully appreciated.

Errors in Base Measurement. The following are the chief *sources of error* in base measurement:

1. Changes of temperature ;
2. Difficulties of making contact ;
3. Variations of the bars or tape from the standards.

The refinements of measurement consist especially in —

a. Standardizing the measuring apparatus, or its comparison with a standard of length.

b. Determination of temperature, or its neutralization by the use of compensating bars.

c. Means adopted for reducing the number of contacts to the fewest possible, and of making these with the greatest degree of precision.

The *inherent difficulties* of measurement *with bars* of any kind are :

1. Necessity of measuring short bases because of the number of times which the bar must be moved.
2. Expense, as a considerable number of men are required.
3. Slowness, the measurement often occupying from a month to six weeks.

The *advantages* of measurement made with a *steel tape* are :

1. Great reduction in the number of contacts, as the tapes are about three hundred feet long as compared with bars of about twelve feet.
2. Comparatively small cost because of the few persons required.
3. Shortness of the time employed, an hour to a mile being an ordinary record in actual measurement
4. Errors in trigonometric expansion may be reduced by increasing the length of the base from 5 miles, the average length of a bar-measured base, to 8 miles, not an uncommon length for tape-measured bases.

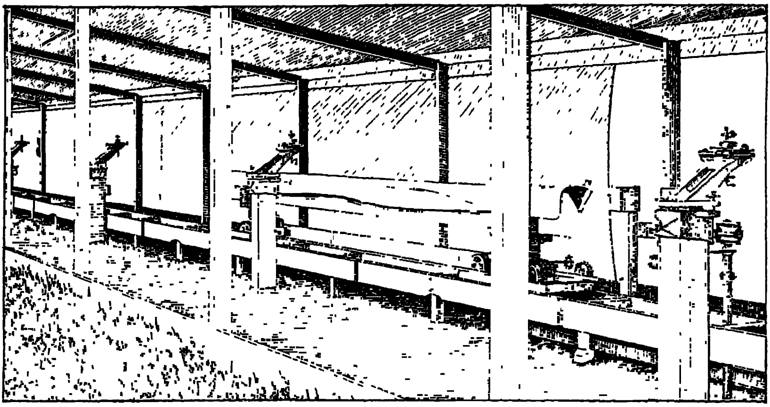


Fig. 120.

Steel tapes offer a means of measuring base lines which is superior to that obtained by measuring bars, because they combine the advantages of great length and simplicity of manipulation, with the precision of the shorter laboratory standards, providing only that means be perfected for eliminating the errors of temperature and of sag in the tape. Base lines can be so conveniently and rapidly measured with long steel tapes as to permit of their being made of greater length than has been the practice with lines measured by bars, and as a result, still greater errors may be introduced in tape-measured bases and yet not affect the ultimate expansion any more than will the errors in the latter, because of the greater length of the base.

The tapes used for this work are of steel, either 300 feet or 100 meters in length. The tapes used by the Coast Survey are 101.01 meters in length, 6.34 millimeters by 0.47 millimeters in cross-section, and weigh 22.3 grams per meter of length.. They are subdivided into 20 meter spaces by graduations ruled on the surface of the tape, and their ends terminate in loops obtained either by turning back and annealing the tape on itself, or by fastening them into brass handles. When not in use, the tapes are rolled on reels for easy transportation.

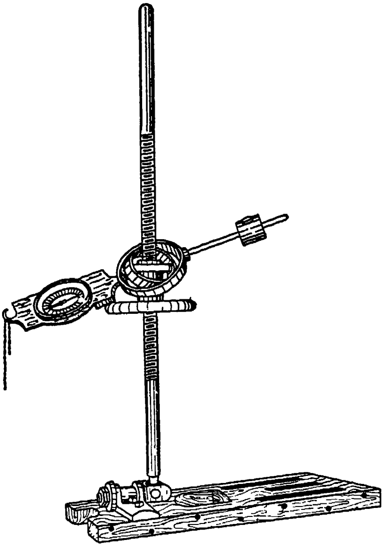
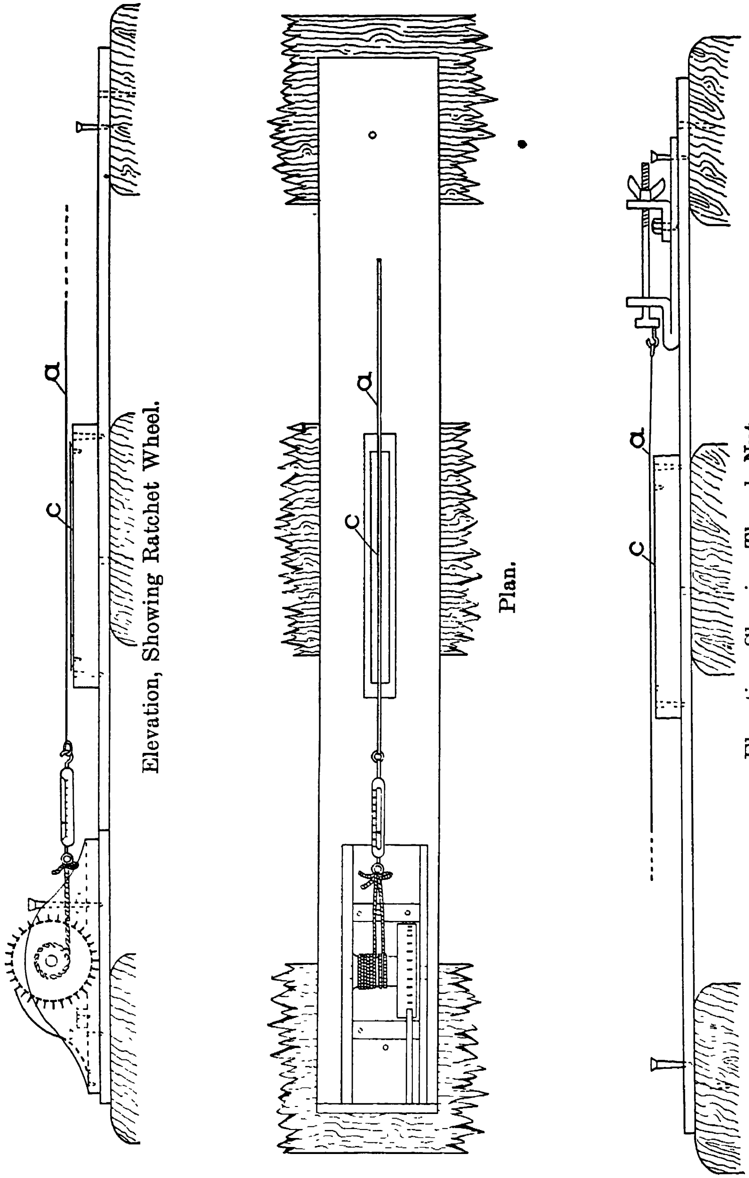


Fig. 121.

The steel tapes used by the Geological Survey are similar to those used by the Coast Survey, excepting in their length, which is a little over 300 feet. They are graduated for 300 feet and are subdivided every 10 feet, the last 5 feet of which at either end is subdivided to feet and tenths. The various instrument-makers now carry such tapes in stock, wound on hand-reels. All tapes must be standardized before and after use, by comparison with laboratory standards, and, if possible, thereafter frequently in the field by means of an iced-bar apparatus.

In measuring with steel tapes, a uniform tension must be applied. In order to get a uniform tension of 20 to 25 pounds, some form of stretcher should be used. That used by the U.S. Coast Survey consists of a base of brass or wood, 2 or 3 feet in length by a foot in width, upon which is an upright metallic standard, and to this is attached by a universal joint, an ordinary spring-balance, to which the handle of the tape is fastened. See Fig. 121. The upright standard is hinged at its junction with the base, so that when the tape is being stretched, the tapeman



Elevation, Showing Ratchet Wheel.

Plan.

Elevation, Showing Thumb Nut.

Fig. 122.

can put the proper tension on it by taking hold of the upper end of the upright standard and using it as a lever, and by pulling it back toward himself he is enabled to use a delicate leverage on the balance and attain the proper pull.

The *thermometers* used are ordinary glass thermometers, around the bubbles of which should be coiled thin annealed steel wire, so that by passing them in the air adjacent to the tape, a temperature corresponding to that of the tape can be obtained. Experience with such thermometers shows that they closely follow the temperature of the steel tape. For the best results, two thermometers should be used, each at about one-fourth of the distance from the extremities of the tape.

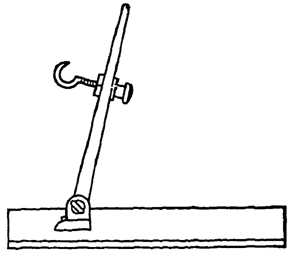


Fig. 123.

The stretching device used by the U. S. Geological Survey is much simpler and more quickly manipulated than that of the Coast Survey. The chief object to be attained in tension is steadiness and uniformity of tension; the simplest device which will attain this end is naturally the best. Two general forms of such devices are employed by the U. S. Geological Survey, one for the measurement of base lines along railways, where the surface of the ties or the roadbed furnishes support for the tape, and the device must therefore be of such kind as to permit of the ends being brought close to the surface; the other is employed in measurements made over rough ground, where the tape may frequently be raised to considerable heights above the surface and be supported upon pegs.

The stretcher used by the Geological Survey for measuring on railways is illustrated in Fig. 122, and was devised by Mr. H L. Baldwin. It consists of an ordinary spring-balance attached to the forward end of the tape, where a tension of twenty pounds is applied, the rear end of the tape being caught over a hook which is held steadily by a long screw with a wing-nut, by which the zero of the tape may be exactly adjusted over the mark scratched on the zinc plate. The spring-balance is held by a wire running over a wheel, which latter is worked by a lever and held by ratchets in any desired position, so that by turning the wheel, a uniform strain is placed on the spring-balance, which is held at the desired tension by the ratchets.

The tape-stretcher used by the U. S. Geological Survey off railways consists of a board about 5 feet long, to the forward end of which is attached by a strong hinge, a wooden lever about 5 feet in length, through the larger portion of the length of which is a slot. See Fig. 123. Through the slot is a bolt with wing-nut, which can be raised or lowered to an elevation corresponding with the top of the hub over which measurement is being made; hung from the bolt is the spring-balance, to which the forward tapeman gives the proper tension by a direct pull on the lever, the weight of the lever and the friction in the hinge being such as to make it possible to bring about a uniform tension without difficulty. The zero on the rear end of the tape is adjusted over the contact mark on the zinc by means of a similar lever with hook-bolt and wing-nut, but without the use of spring-balance.

Laying out the Base. The most laborious operation in base measurement is its preliminary preparation, which consists of:

1. Aligning with transit or theodolite;
2. Careful preliminary measurement for the placing of stakes on rough ground;
3. Placing of zinc marking-strips on the stakes.

Base lines measured with steel tapes across country are aligned with transit or theodolite, and are laid out by driving large hubs of 3 × 6 scantling into the ground, the tops of the same projecting to such a height as will permit a tape-length to swing free of obstructions. These large hubs are placed by careful preliminary measurement at exact tape-lengths apart, and between them as supports, long stakes are driven at least every 50 feet. Into the sides of these near their tops are driven horizontally, long nails, which are placed at the same level by eye, by sighting from one terminal hub to the next. The tape rests on these nails and on the surface of the terminal hubs are tacked strips of zinc on which to make the contact marks. A careful line of spirit-levels must be run over the base-lines, and the elevation of the hub or contact-mark of each tape-length must be determined in order to furnish data for reduction to the horizontal.

In measuring over rough ground, six men are necessary: two tape-stretchers, two markers, two observers of thermometers. one of whom will record. The co-operation of these men is obtained by a code of signals, the first of which calls for the application of the tension; then the two tape-stretchers by signal announce when the proper tension has been applied; then the rear observer adjusts the rear graduation over the determining mark on the zinc plate and gives a signal, upon hearing which, the thermometer recorder near the middle of the tape lifts it a little and lets it fall on its supports, thus straightening the tape. Immediately thereafter the front observer marks the position of the tape graduation on the zinc plate, and at the same time the thermometers are read and recorded.

After the measurement of the base line has been completed in the field, the results of the measurement have to be reduced for various corrections, among which are:

Comparison with standard measure;
Corrections for inclination and sag of tape if such is used;
Correction for temperature.

The first correction to be applied is that of reducing the tape-line to the standard, "standardizing" the tape as it is called. By sending the tape to the National Bureau of Standards at Washington, D. C., a statement may be had of the length of the tape compared with the standard. For this service a small fee is charged. For an additional fee a statement may be had of the temperature and pull at the ends for which the tape is a standard.

As the length of a steel tape varies with the temperature, one of the most uncertain elements in the measurement of a base with the steel tape, is the change in the length of the standard due to changes of temperature. Corrections, therefore, must be made for every tape-length as derived from readings of one or more thermometers applied to the tape in the course of measurement.

Steel expands .0000063596 of its length for each degree Fahrenheit. This decimal multiplied by the average number of degrees of temperature above or below 62 degrees at the time of the measurement, gives the proportion by which the base is to be diminished or extended on account of temperature changes. This correction is applied usually by obtaining with great care, the mean of all thermometer readings taken at uniform intervals of distance during the measurement.

The data for the correction for inclination of base are obtained by a careful line of spirit-levels over the base-line. In the course of this leveling, elevations are obtained for every plug upon which the tape rests. The result of this leveling is to give a profile showing rise or fall in feet or fractions thereof between the points of change in inclination of the tape-line. From this and measured distances between these points, the angle of inclination is computed by the formula

$$\sin \theta = \frac{h}{D}$$

In which D is the length of the tape or measured base:
and h is the difference in height of the ends of tape or measured base, expressed in feet.
θ is the angle of slope expressed in minutes.
The correction in feet to the distance is that computed by the equation

$$\text{Correction} = D \frac{\sin^2 \ 1'}{2} \theta^2$$

An *approximate* formula for reducing distances measured upon sloping ground to the horizontal is expressed by the rule: Divide the square of the difference of level by twice the measured distance, subtract the quotient thus found from the measured distance, and the remainder equals the distance required ; thus

$$d = D - \frac{h^2}{2D}$$

in which d equals the horizontal or reduced distance.

When the base measurement is made with steel tape across country, and accordingly is not supported in every part of its length, there will occur some change in its length, due to sag. As previously explained, the tape should be rested upon supports not less than 50 feet apart. With supports placed even this short distance apart, however, a change of length will occur between them, while even greater changes will occur should one or more supports be omitted as in crossing a road, ravine, etc. Since tapes are standardized by laying them upon a flat standard, it is necessary to determine the amount of shortening from the above causes.

The following reduction formulæ apply :

Let w = weight per unit of length of tape :
t = tension applied
$a = \frac{w}{t}$
n = number of sections into which tape is divided by supports.
l = length of any section
L = normal length of tape or right-line distance between n marks when under tension : $= nl$ approximately.

If a tape be divided by equidistant supports, the difference in distance between the end graduations, due to sag, or the correction for sag = dL becomes

$$dL = \frac{1}{24} a^2 (n_1 l_1^3 - n_2 l_2^3)$$

If one or more supports are omitted, then the omission of m consecutive supports shortens the tape by

$$\frac{1}{24} m (m + 1) (m + 2) a^2 l^3 :$$

when l is the length of the section when no supports are omitted.

Example. Let $n = 6$; $l = 50$ feet ; $w = .0145$ = weight in pounds per foot found by dividing whole weight of tape by whole length ; $t = 20$ pounds.

$$d = \frac{nl}{24} \times \left(\frac{wl}{t}\right)^2 = 0.0164 \text{ feet}$$

which is the amount of shortening of each tape-length. This correction is always negative.

If there had been 86 full tape-lengths in measured base-line, the total corrections for sag would be 86×.0164=1.413 feet.

THE PLANE-TABLE.

Construction. The plane-table consists essentially of a drawing-board mounted upon a tripod. This board is usually twenty-four by thirty inches, constructed in sections to prevent warping; it is attached to the tripod by a three-screw leveling base arranged to permit the board to be turned in azimuth and to be clamped in any position.

The instrument is designed to at once sketch in the field, to scale, the lengths and relative directions of all lines and the positions of objects to be included in the survey. For drawing straight lines, a steel ruler is provided upon which is mounted at each end, a pair of open sights like those of the compass, or, a telescope is mounted at the center of the ruler, fitted with stadia wires, a vertical arc and a longitudinal striding level. The eyepiece should be inverting, and whether the open sights or the telescope is used, the line of sight should always be parallel to the edge of the ruler. The straight edge with the attached telescope or open sights is called the *alidade.*

For leveling the instrument, two cylindrical levels, at right angles to each other, are mounted upon the alidade and either an attached or detached compass is provided for determining the bearing of lines.

For attaching the paper to the board, various devices are used. One consists of a roller at each end of the table upon one of which the paper is wound up as it is unrolled from the other, the edges of the paper being held close to the board by spring clips. This arrangement permits the paper to be used in a continuous roll and to be tightly stretched over the board. The use of the continuous roll of paper is undesirable, however, and separate sheets of proper size should be used, attached to the board and held firmly in place by the spring clips provided with the instrument. The use of thumb-tacks should be avoided.

Under the most favorable conditions, the plane-table is a very awkward instrument and difficult to handle, but it is admirably adapted to filling in the details of a topographical survey. For this purpose it is the standard instrument of the United States Geodetic Survey and is also largely used by the United States Geological Survey. It cannot be used on damp or very windy days and is not therefore, of as general utility as the transit and stadia.

Fig. 124 shows one form of construction of the plane table with leveling screws and Fig. 124*a* shows a plane table with a much simpler form of leveling head. This latter was designed by Mr. W. D. Johnson and has received the approval of the topographers of the United States Geological Survey. The whole arrangement is very light, but does not permit of as close leveling as does the usual form with leveling screws.

Adjustments.

1st. *To determine whether the edge of the ruler is straight.*

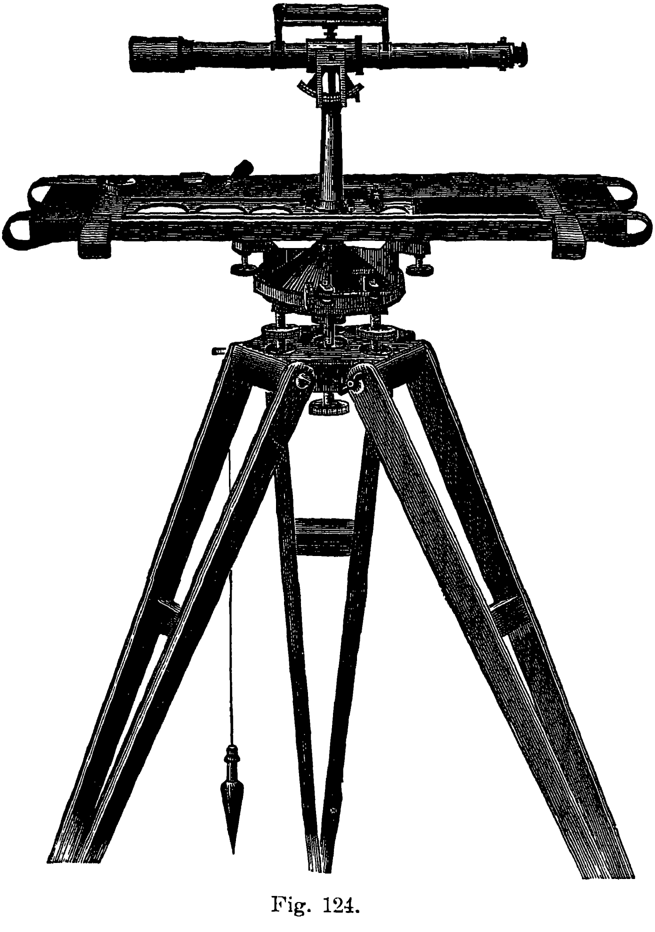


Fig. 124.

Place the ruler upon a smooth surface, and draw a line along its edge, and also lines at its ends. Reverse the ruler on these lines, and draw another line along its edge. If these two lines coincide, the ruler is straight.

2nd. *To make the plane of the table horizontal when the bubbles are in the center of the tubes.* Assuming the table to be plane, set the alidade in the middle of the table, level by means of the leveling screws, draw lines along the edge and ends of the ruler, and reverse the alidade on these lines. If the bubbles remain in the center of the tubes, they are in adjustment. If they do not, correct one-half of the error by means of the leveling screws and the remainder by means of the capstan-headed screws of the level tubes. Repeat the operation until the bubbles remain in the center of the tubes in both positions of the alidade.

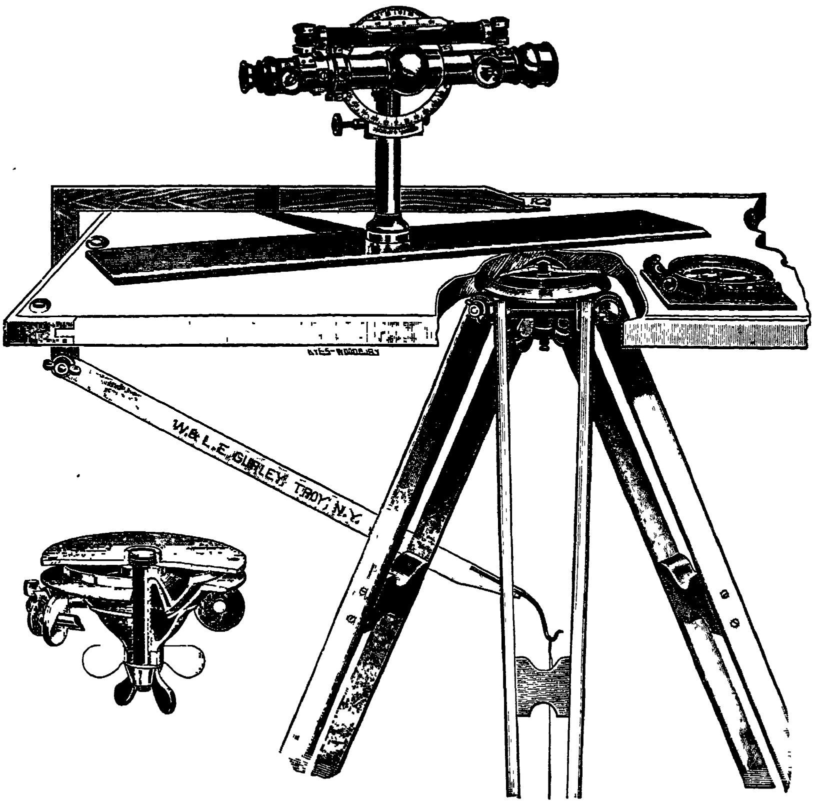

W.& L.E. GURLEY TROY, N.Y.

Fig. 124*a*.

3rd. *To make the line of collimation perpendicular to the horizontal axis of the telescope.*

Level the table and point the telescope towards some small and well-defined object. Remove the screws which confine the axis of the telescope in its bearings, reverse the telescope in its bearings, that is, change the axis end for end, being careful not to disturb the position of the alidade upon the table, and again sight upon the same object. If the intersection of the cross hairs bisects the object, the adjustment is complete. If not, correct one-half of the error by means of the horizontal screws attached to the reticle. Sight on the object again and repeat the operation until the line of collimation will bisect the object in both positions of the telescope.

4th. *To make the line of collimation parallel to the axis of the bubble tube.*

Attach the longitudinal striding level to the telescope and carry out the adjustment by the "peg" method as described for the transit.

5th. *To make the horizontal axis of the telescope parallel to the plane of the table.*

Level the table and point the telescope to a well-defined mark at the top of some tall object, as near as possible consistent with distinct vision. Turn the telescope on its horizontal axis, and point to a small mark at the base of the same object. Draw lines on the table at the edge and ends of the ruler. Reverse on these lines, point the telescope to the lower object and turn the telescope upon its horizontal axis. If the line of collimation again covers the higher point, the adjustment is complete. If it does not, correct one-half of the error by means of the screws at one end of the horizontal axis.

6th *To make the vertical arc or circle read zero when the line of collimation is horizontal.*

Level the table and measure the angle of elevation or depression of some object. Remove the table to the object, level as before, and measure the angle of depression or elevation of the first point Half the difference, if any, of the readings is the error of the adjustment. Correct this by means of the screws attached to the vernier plate, and repeat the operation until the angles as read from the two stations are equal.

Use. The plane-table is used for the immediate mapping of a survey made with it, no angles being measured, but the direction and length of lines being plotted at once, upon the paper. The simplest case is the location of a number of points from one central point, called the method of radiation. The table is "set up" so that some convenient point upon the paper is over a selected spot upon the ground and is then clamped in azimuth. Mark the point upon the table by sticking a needle into the board. Now bring the edge of the alidade in contact with the needle and swing it around until the line of sight, which is parallel to the edge of the ruler, is directed to the point to be located. Having determined the scale of the plat, a line is drawn along the edge of the ruler to scale, equal to the distance to the desired point, such distance having been measured either with the tape or stadia. In the same way locate all of the other points, which may include houses, trees, river banks, etc. If the plane-table is set up in the interior of a field at a point from which all of the corners are visible, the corners can be thus located and after being connected, there results a plot of the area. Instead of occupying a point in the interior of the field, one corner may be selected from which all of the others are visible, or a point outside of the field may be chosen from which to measure the lines to the several corners. Evidently from such a survey, data is lacking from which to calculate the area, and either the map must be scaled for additional data or the area measured with the planimeter.

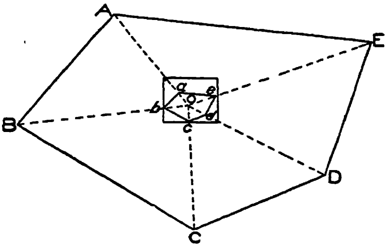


Fig. 125.

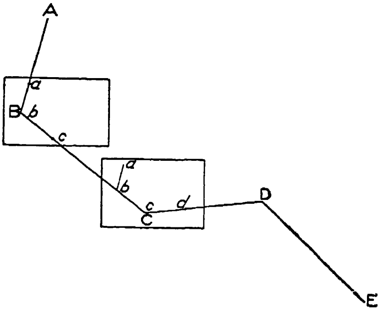


Fig. 126.

The Fig. 125 illustrates the method of surveying a closed *o* and drawn to an exaggerated scale. The area *abcde* representing to scale, the area ABCDE. It may be desirable to set up the table at some other point, as for instance one of the corners of the field, and run out some of the lines to the other corners as a check upon the work.

Traversing, or the Method of Progression. This method is practically the same as the method of surveying a series of lines with the transit, but requires that all of the points be accessible. It is the best method of working as it provides a complete check upon the survey.

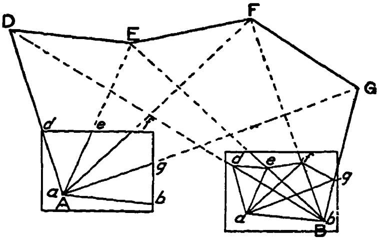


Fig. 127.

Let ABCDE, Fig. 126, be the series of lines to be surveyed by *traversing*. Set up the table at B, the second angle of the line. so that the point *b* upon the paper will be directly over the point B upon the ground. (The point *b* should be so chosen as to leave room upon the paper for as much of the traverse as possible.) Stick a needle at the point *b* and place the edge of the alidade against it. Swing the alidade around until the line of sight covers the point A. Measure BA and lay it off to the proper scale as *ba*. Now turn the alidade around the point *b* and sight to and measure the distance BC and plot it to scale as *bc*. Remove the instrument to *c* with the point *c* upon the paper directly over C upon the ground, and *cb* in the direction of CB. This is difficult to accomplish with the plane-table, but if the plot is drawn to a large scale, it must be done. If the plot is drawn to a small scale, it will be sufficiently accurate to set the table over the point C as nearly as possible in the proper direction and then turn the *board* in azimuth until *b* is in the direction of B. Stick a needle at *c* and check the length of *cb*. Swing the alidade around *c* until the line of sight covers D, measure CD and plot *cd*. Remove to D and proceed as before and so on through the traverse.

If the survey is of a closed field, the accuracy of the work will be checked by the closure of the survey.

The method of progression is especially adapted to the survey of a road, the banks of a river, etc., and often many of the details may be sketched in with the eye.

When the paper is filled, put on a new sheet, and on it, fix two points, such as D and E, which were on the former sheet and from them proceed as before. The sheets can afterward be united so that all points on both shall be in their true relative positions.

Method of Intersection. This is the most rapid method of using the plane-table. Set up the instrument at any convenient point, as A in Fig. 127 and sight to all the desired points as D, E, F, etc., which are visible, and draw indefinite lines in their directions. Measure any line as AB, B being one of the points sighted to, and plot the length of this line upon the paper to any convenient scale. Move the instrument to B so that *b* upon the paper will be directly over B upon the ground, and so that *ba* upon the paper will be in the direction of BA upon the ground as explained under the method of progression. Stick a needle at the point *b* and swing the alidade around it, sighting to all the former points in succession, and draw lines in their direction. The intersection of these two sets of lines to the several points will determine the position of the points. Connect the points as *d*, *e*, *f*, *g*, in the figure. In surveying a field, one side may be taken as the base line. In choosing the base line, care must be exercised to avoid very acute or obtuse angles: 30° and 150° being the extreme limits. The impossibility of always doing this, sometimes renders this method deficient in precision.

TOPOGRAPHICAL SURVEYING.

A topographical map is one showing the configuration of the surface of the ground of the area to be mapped and includes lakes, rivers, and all other natural features, and sometimes artificial features as well.

A topographical survey is one conducted for the purpose of acquiring information necessary for the production of a topographical map of the area surveyed.

Nearly all engineering enterprises involve a topographical importance of the contemplated work. The construction of an important building may involve a survey of the foundation site to determine the amount of cut and fill; the construction of a bridge will involve a hydrographic survey of a body of water to acquire information in regard to direction and velocity of current, depth of water, nature of bottom, and proper site for piers and abutments. A proposed railroad will not only involve a survey of the line itself, but a topographical survey extending from 200 to 400 feet upon each side. The design of a sewer system or a waterworks system, dams, reservoirs, canals, irrigation channels, tunnels, etc, all involve topographical surveys.

In what follows it is intended to outline the methods of conducting field operations, based partly upon the nature and importance of the problem involved, and partly upon the instruments used. The different methods of representing topography and the involved drafting-room work will be fully treated in Topographical Drawing.

The field operations, in so far as the methods and instruments are concerned, may be classified as follows:

1. Sketching by the eye, without or with the tape for measuring distances.
2. Sketching with the aid of the Locke hand-level or clinometer, horizontal distances being measured either by pacing or with the tape.
3. Determining the elevation of points with the wye-level, horizontal distances being determined either with the stadia or tape.
4. Determining points with the transit and stadia.
5. Topographical sketching with the plane-table and stadia.
6. Photography.
7. Triangulation.

It is evident that the first method is entirely lacking in accuracy, and such work should be done only when speed is the most important consideration, only the roughest approximation to the topographical features being attempted; contour lines cannot be located. Work of this nature is of value principally for purposes of promoting an enterprise; artistic, showy plates being desired. Little can be said descriptive of the manner of carrying out the field work, since this will require considerable artistic ability as well as the ability to "see" things and estimate distances. Comparatively few men possess the ability to carry out topography of this nature It necessarily follows that the work must be done entirely by sketching in the field, and for this purpose the following equipment is needed:

2 or 3 medium pencils, kept well sharpened.
Rubber eraser.
Thumb-tacks.
Several sheets of drawing paper, 14″ × 14″.
One light drawing board, 15″ × 15″.

A pocket compass will be useful in determining the bearing to prominent objects to tie in the stations of the survey. A Locke hand-level or Abney clinometer will also be useful for finding approximate heights, and either of these instruments can be readily carried in the pocket. It will be more convenient to have the paper cross-ruled into one-fourth inch squares, the center line being ruled in red, but if drawing paper is used, it will be necessary to add an engineer's scale to the equipment. The back of the drawing board should be fitted with a leather pocket, with flap and button, in which the blank sheets and the finished topographic sheets should be kept. A strap attached to the board and to go over the shoulder, will prove a great convenience. A waterproof cover should be provided to protect the board and sheets in case of rain.

A compass or transit survey forms the backbone of the topography, and the sketching should include an area upon each side of the line so surveyed, and running parallel with it.

A separate sheet should be used for each course (by course is intended the straight line from one turning point to the next), no matter how short it may be. Begin at the bottom of the sheet and sketch the topography up the sheet, that is, in the direction of the progress of the survey, and number the sheets in order. Begin each new sheet with the same station that ended the preceding sheet. After the field work is completed, the sheets can be laid down in order, the angles between their center lines corresponding to the deflection angles as given by the transit notes of the survey. The topography can now be traced upon tracing cloth in a continuous sheet. The method above outlined will result in a saving of time, especially in working up the topographic plat.

The second method commends itself in connection with a preliminary survey of a highway, steam or electric road, irrigation channels, canals, etc. The equipment should be as follows:

1 or 2 straight edges, about 12 feet in length.
1 or 2, 100-foot steel tapes.
1 or 2 plumb-bobs.
1 pocket compass.
2 or 3 medium pencils, kept well sharpened.
Rubber eraser.
Thumb-tacks.
Several sheets of drawing paper or cross-section paper, 14″ × 14″.
One light drawing board, 15″ × 15″ with waterproof cover.

The topographic party should be made up of the topographer and one or two assistants, depending somewhat upon the nature of the survey and the country traversed. If the country permits of rapid progress of the transit and level party, two assistants will be necessary to keep the topography abreast of the survey. Rapid work may, however, be done with one assistant, provided the topography does not extend more than 200 feet each side of the transit line.

The Abney clinometer is well adapted for this class of work, on account of its portability, which is an important item in a rough country with steep side slopes. It can be used in the same way as the Locke hand-level, if necessary, but is a more generally useful instrument, as is described in Part 1. The straight edge should be of well-seasoned, straight-grained material, as light as possible, but so constructed as to prevent warping. It should be divided into spaces of one foot each, painted alternately red and white. The tapes should be of band steel, as they are subjected to rough usage, and they should be divided to feet and tenths at least. A plumb-bob is necessary for plumbing down the end of the tape on steep slopes. The pocket compass is a *necessary* adjunct in work of this character. The drawing paper should preferably be cross-section paper ruled into one-fourth inch squares with a heavy center line in red, but if ordinary drawing paper is used, it will be necessary to include in the outfit an engineer's scale, by means of which distances may be platted upon the sheet. Enough of these sheets should be carried to cover a day's work, but no more. The drawing board should be fitted up as described under the previous method.

Method of Procedure. The transit line furnishes, of course, the proper distance upon each side of this line, by locating points both as to distance and elevation, upon perpendiculars from the transit stations. In rough country, it may be necessary to locate these points intermediate between the transit stations. Before starting out upon a day's work it is necessary to procure from the level party, the elevation of the transit stations, or if the topography keeps pace with the transit survey, the elevation may be obtained from the leveler at each station. For points intermediate between transit stations, the elevations may be gotten closely enough with the clinometer or hand-level. The number of each station as well as its elevation, should be noted upon the topographic sheet, and the topography will include the location of contour lines, at proper vertical intervals, as well as all streams, lakes, property lines, etc. An example showing the method of keeping the field notes, will at the same time best serve to explain the methods of conducting the survey.

Beginning with station 0 at the bottom of the sheet, the number and elevation of the station are noted. See Fig. 128. Sending the assistant out upon one side of the transit line and at right angles thereto, he holds the rod at points to be designated by the topographer, the distances to be determined by pacing, or with the tape, and the elevations determined either by sighting upon the rod with the clinometer, or by laying the straight edge upon the ground at right angles to the line and applying the clinometer to it to determine the slope, from which elevations can at once be determined. Contour points are then readily interpolated and the distance out platted to scale upon the sheet and a note made of the elevation of the contour lines. If a lake or stream intervenes within the limits of the topographic survey, determine the distance to and elevation of the shore line and plat upon the sheet. Determine points upon the other side of the transit line in the same way.

If one or more contour lines cross the transit line between stations, determine the points of crossing and plat the points upon the sheet, to scale, as shown between stations 0 and 1. It will be noticed in this case that the elevation of station 0, is 138 feet and of station 1, is 141 feet. If contours are to be taken at vertical intervals of five feet, it is apparent that the 140-foot contour line

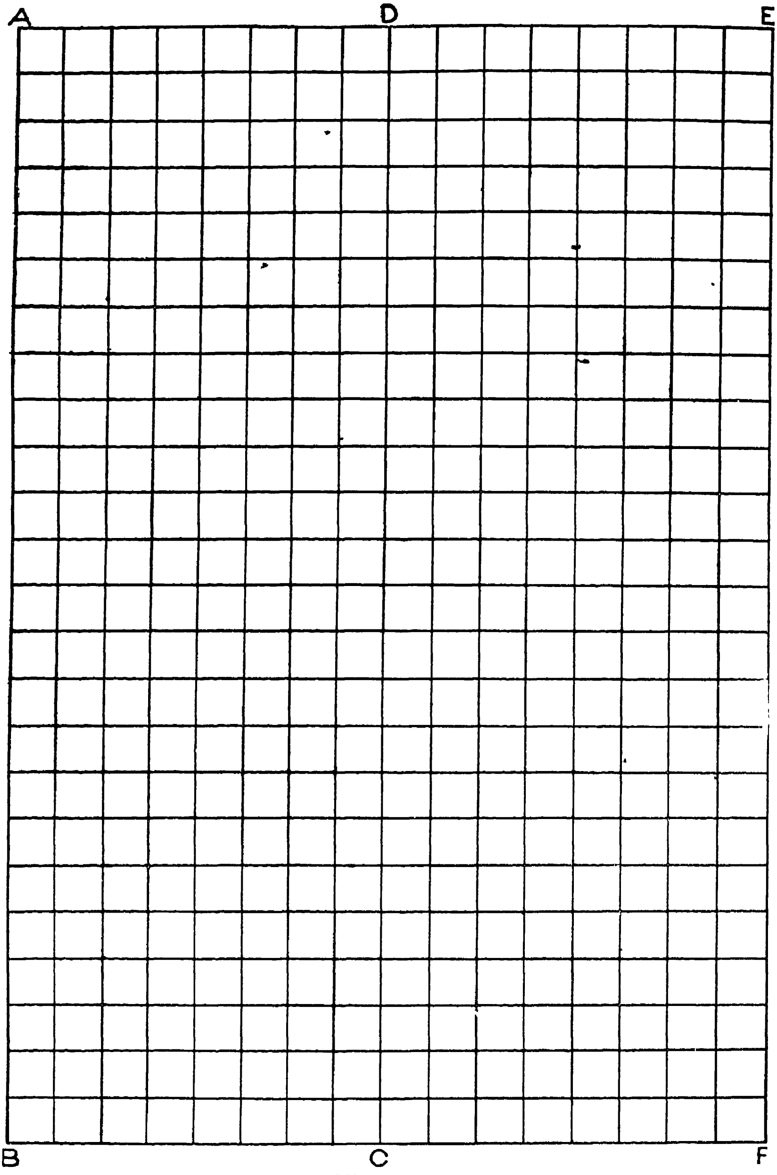


Fig. 128.

the ground is uniform, the point of crossing may be taken at two-thirds of the distance from 0 to 1. Otherwise, locate the point with the clinometer.

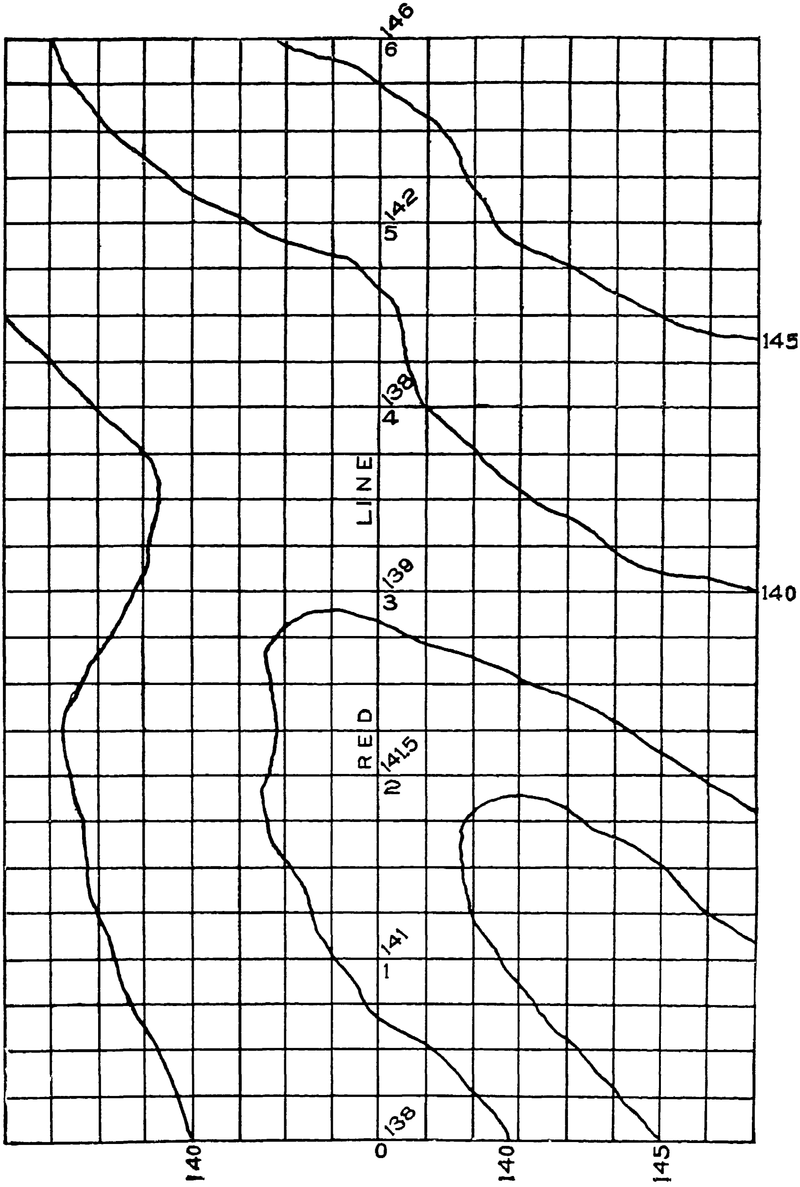


Fig. 128.

Now go to station 1 and locate contour points and other same contour line, sketching in the curve of the line with the eye. Use a separate sheet for each portion of the transit line from turning point to turning point; this will require that the turning points appear upon two consecutive sheets. Likewise, if the length of the line between turning points is too long to be platted upon a single sheet, begin the second sheet with the same station that completed the first sheet and so continue throughout the survey. As each sheet is completed, number it and return to the pocket on the back of the drawing board. The pocket compass should be used for determining the bearing of property lines, roads, streams, etc., crossed by the survey, and to take the bearings to prominent objects.

The topographic sheets should be filed away in such a manner as to make them easily accessible at any time, as the engineer in charge of the transit survey may wish to consult them from time to time. The office work of preparing the topographic plat can be very expeditiously carried out as before described.

The use of the wye-level as a topographic instrument is limited, but for certain kinds of work the instrument is the most satisfactory, as for instance, the survey of a dam-site; the survey of a reservoir-site; the survey of a town preparatory to planning sewer and waterworks systems and the planning of street pavements.

The instrument should be fitted with stadia wires for measuring horizontal distances, and this will usually prove a great convenience, resulting in saving of both time and expense. A steel tape should, however, be included in the equipment for field work, for the purpose of checking measurements with the stadia. In addition to the above there should be provided, the following equipment:

Self-reading level rod, capable of being read to hundredths of a foot.
Hatchet.
Marking crayon.
2 or 3 medium pencils, kept well sharpened.
Plumb-bob.
Rubber eraser.
Portable turning point.

The method of using the level rod in connection with the stadia for measuring distances has been fully discussed in Part II.

The portable turning point will prove of great convenience and may be made from a triangular piece of thin steel, with the corners turned down to project about one inch.

If the level is to be used with the tape, the party will be made up of the levelman, two tapemen and a rodman, unless the nature of the work will permit of the rodman carrying the rear end of the tape. If the stadia is used for measuring distances, only the rodman will be required in addition to the levelman. The levelman carries the note book and enters into it all rod readings both for elevation and distances. These notes should be entered upon the left-hand page, the right-hand page being reserved for notes and sketches, which should be as full as possible. The levelman should cultivate the practice of calculating the elevations of the stations as the work progresses, at least of the turning points and bench-marks, in order that the results may be checked and errors discovered at once and corrected. If this work is left to be afterward carriod out in the office, errors may be discovered that may require considerable time to locate and correct.

If the area to be surveyed is, for instance, a reservoir site, it will be found most convenient to cover the area with a system of rectangles as shown in the figure, the parallel lines being spaced from 200 to 400 feet apart as may be most desirable. These lines should be run in with the transit, stakes being set at the intersections of the cross lines, or if the area is not very extended and is comparatively level, by means of the level itself, the perpendicular distances between the parallel lines being measured with the tape.

These lines having been laid down, the next step is to establish a system of bench-marks over the area. Begin by establishing a "standard" bench-mark at some central point upon a permanent object, easily identified, and from thence radiate in all directions, returning finally to the original bench-mark for purposes of checking. Having located and satisfactorily checked the bench-marks, begin by running the level over all the lines running in one direction, as from A to B, back from C to D and so on, taking rod readings at every fifty or one hundred feet, in addition to the readings at the stakes at intersections of cross lines. It is to be points. Next run the level over the lines at right angles to the former ones and in the same way, checking the levels at intersections. Advantage should be taken of every opportunity to check upon bench-marks previously located, and to establish others.

In keeping the field records, the notes of the two sets of lines should be kept in separate books ; that is to say, if, for instance, one set of lines run north and south, and, therefore, the other east and west, the notes of the north and south lines should be entered in one set of books and the notes of the east and west lines in another set, and a note should be made of the direction in which a line is run, as from north to south or from east to west.

In conducting a survey for the preparation of a topographical map necessary to the design of a sewer or waterworks system, much the same method is to be followed, but now the streets and alleys take the place of the rectangular system referred to above. As before, all the streets and alleys running in parallel directions are to be gone over in a systematic way, readings being taken fifty or one hundred feet apart in addition to street and alley intersections. (By street and alley intersections is intended the intersections of the center lines, the lines of levels being run along these center lines.) If a fairly accurate map of a town is available, the distances measured with the tape along the center line of the streets and alleys will serve as a check upon the map. If, however, discrepancies occur or there is no map available, it will be necessary to use the transit for staking out street lines and for determining the relative directions of these lines. It follows that the topography of the ground between streets and alleys can only be approximated, but sufficient points accurately determined will have been established to permit the platting of a contour map, from which the system can be laid down.

The office work involved in the survey of an area, as above described, consists in preparing profiles of the level lines and preparing a plat of the lines surveyed. From the profiles the contour points can be laid down in their proper position upon the plat, and as each point is laid down, its elevation should be noted in pencil, and after all the points have been platted, the points in the same contour line can be connected—preferably free-hand—producing the contour map. The scale to be adopted will depend upon the nature of the work, but should be as large as possible, consistent with the convenient handling of the map.

Transit and Stadia. The method by transit and stadia is of more general application than the preceding method, points being located by "polar co-ordinates," that is to say, by direction and distance from a known point, the elevation being determined at the same time.

Method of conducting field operations. If the area to be surveyed is small, the preceding method, based upon a system of rectangles, will prove satisfactory, and the elevations of the corners and salient points can be determined at the same time that the lines forming the rectangles are being laid down. Especial care should be taken to check the elevations of the corners.

In making a survey for a sewer or a waterworks system, the transit and stadia method will be found efficient, especially in cases where no survey has previously been made, the map, if it exists at all, having been compiled from the records in the County Recorder's office. The bench-marks necessary in a survey of this kind, however, should be established with the wye-level, and it may be desirable to determine the elevation of street intersections in the same way.

If the area to be surveyed is too large, or of uneven topography, proceed as follows: Choose a point, as the intersection of two streets, the corner of a farm, or an arbitrary point conveniently located and drive a stake firmly at this point, "witnessing" it from other easily recognized points or stakes. The transit should be set over this point with the vernier reading zero, and the instrument pointed by the lower motion in the direction of the meridian. This may be the true meridian previously determined, the magnetic meridian as shown by the needle, or an arbitrary meridian assumed for the purpose of the survey. It will generally be more satisfactory to run out a true meridian by means of the solar attachment, but in any event the direction of the line taken as a meridian should be defined by stakes, firmly driven into the ground, and "witnessed" by stakes or other objects easily recognized. The elevation of the starting point, if not known, is assumed and recorded in the notebook. A traverse line should now be run, its position and direction chosen with a view to obtaining from each station the largest possible number of pointings to salient features of the area under survey, and these pointings are taken while the instrument is set at any station, and before the traverse is completed.

The length of each course is measured with the stadia, and together with the azimuth and the vertical angle, it should be recorded in the notebook. The length, azimuth, and vertical angle of each course should be read from both ends to serve as a check. The additional pointings taken from each course of a traverse are usually called "side shots", and for each there are required the distance, azimuth, and vertical angle. These will locate the point and determine its elevation.

The method of using the stadia has already been quite fully discussed in Part II., and need not be repeated.

The points selected for side shots should be such as will enable the contours to be platted intelligently and accurately upon the map of the area under survey. They should be taken along ridges and hollows and at all changes of slope. They should be taken at frequent intervals along a stream to indicate its course, or along the shore of a lake. It is usually required that the location of artificial structures, such as houses, fences, roads, etc., be determined that they may be mapped in their proper position. Pointings, therefore, should be taken to all fence corners and angles, and to enough corners and angles of buildings, to permit of their being platted. Sufficient points should be taken along roads to determine their direction. Wooded lands, swamps, etc., may be indicated by pointings taken around their edges. In addition to the notes above described, the recorder should amplify the notes with sketches, to aid the memory in mapping.

The traverse, of course, forms the backbone of such a survey, and the accuracy of the resulting topographical map will depend upon the degree of care bestowed upon running the courses. Over uneven ground, it is often desirable to run a secondary traverse from the first, for the more rapid and accurate location of points.

The organization of a party will depend upon the nature of the country traversed and of the results required. Changes in the make-up of parties, as given below, will suggest themselves for any special work.

For economy and speed, the party for taking topography with transit and stadia will consist of a transitman or observer, a recorder in charge of the notebook, who should be capable of making such sketches as are necessary, and two to four men with stadia rods. The greater the distances to be traversed by the stadia men between points taken, the greater number the observer can work to advantage. One or two axemen may be employed if clearing is to be done.

The party may be reduced to two men—one to handle the instrument, record notes and make sketches, the other to carry the rod.

The Plane Table and Stadia. The plane table is an instrument intended for topographic purposes only and is used for the immediate mapping of a survey made with it, no notes of angles being taken, but the lines being platted at once upon the paper. The use of the plane table has been fully described. In topographical work over an extended area, it may be used for filling in details, based upon a previous traverse made with a transit, or based upon a system of triangulation as will be described. Over small areas, the traverse itself may be run with the plane table and the details filled in at the same time. It is the standard instrument of the United States Geological Survey and is largely used upon the United States Geodetic Survey.

The points in favor of the plane table are: Economy, since the map is made at once without the expense of notes and sketches; and as the mapping is all done upon the ground to be represented, all of its peculiarities and characteristics can be correctly represented.

On the other hand, the plane table is an instrument useful only for taking topography; the rodmen are idle while the mapping is being done; the instrument is more unwieldy than the transit, particularly upon difficult ground; the record of the work for a long period is constantly exposed to accident; the distortion of the paper with the varying dampness of air, introduces errors in the map; while the area exposed makes it too unstable to use in high winds

The organization of a party for the taking of topography, using the plane table, is much the same as with the transit and stadia; however, on account of the weight of the instrument, means of transportation must be provided.

A less number of rodmen can be employed than with the stadia, owing to the time required for mapping.

An observer, a man to reduce stadia notes and sketch topography around points determined by intersection or stadia from the plane table station, and one rodman, will make the minimum working party, in addition to which, axemen and a team for transportation will be required.

Photography. The following is taken from Gillespies Surveying (Staley).

"Photography has long been successfully employed by European engineers, notably those of Italy, for the purpose of taking topography. The Canadian Government has also employed it successfully in the survey of Alaska.

The recommendation of this method is the great saving of time in the field, while giving topographic features with all the accuracy required for maps to be platted on a scale of 1 to 25,000.

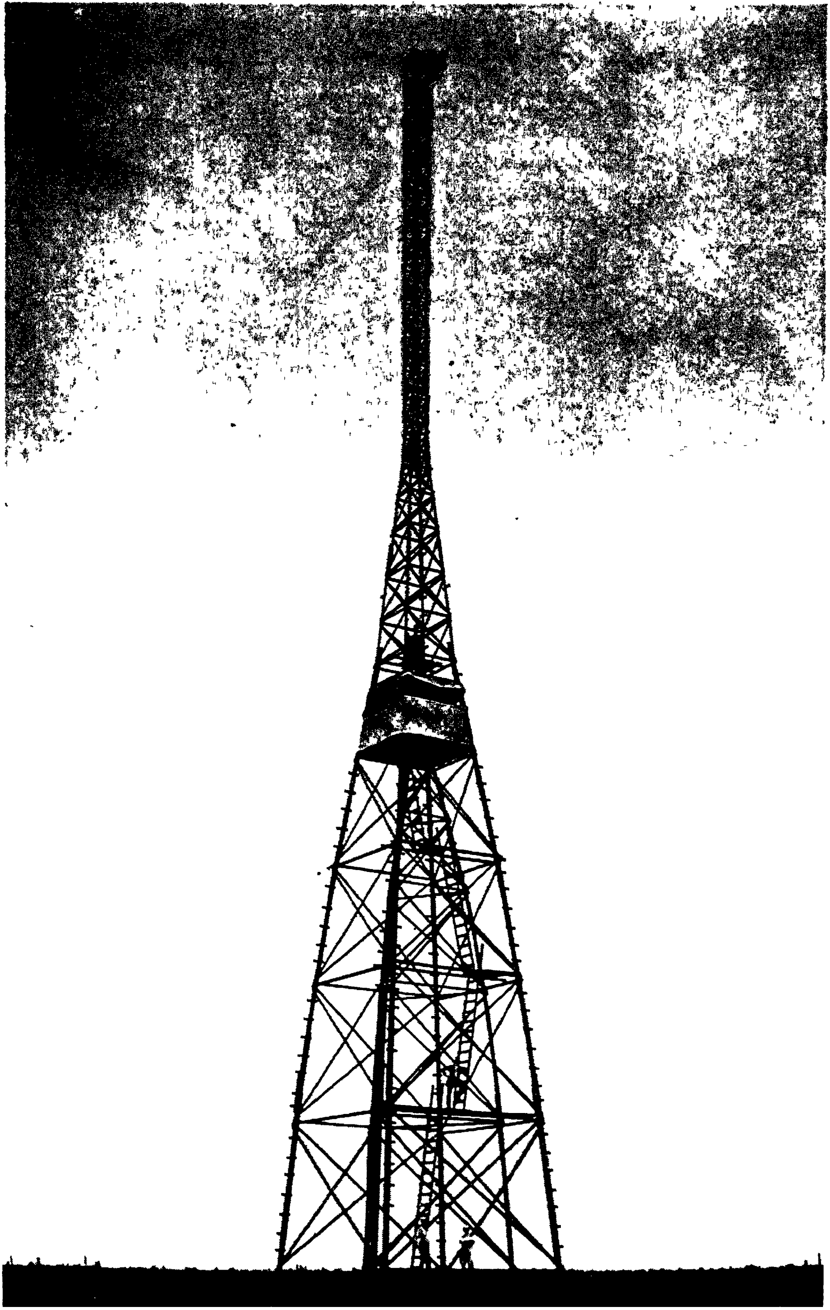
M. Javary states that the maximum error both for horizontal distances and elevations, using a camera with a focal length of twenty inches and a microscope in examining the points, was only 1 in 5,000 as deduced from a number of cases.

M. Laussedat, in his work, found that this method did not require more than one-third the time necessary by the usual methods.

This makes it especially suitable in all mountainous regions, where so much time is lost in getting to and from stations, that little is available for observations and sketching.

A single occupation of a station with photographic apparatus would suffice to complete work that with the ordinary methods would require several days."

Instruments. The ordinary camera may be used, if it is provided with a level. A tripod head for leveling the instrument, and a roughly graduated horizontal circle for reading the direction of the line of sight, when photographing different parts of the horizon, are convenient attachments



A camera is sometimes used upon a plane table, the record of the work being made upon the paper in connection with a set of radial lines drawn from the point representing the station occupied.

Many special forms of instrument combining the camera and theodolite have been devised, some one of which should be used if work of this kind is to be undertaken on a large scale. For a description of these instruments, and a complete treatise on this subject, comprising a discussion of the requirements of the apparatus, the fundamental principles of photography, methods of field work, forms of notes, reduction of notes and making of the map, together with the bibliography of the subject, see United States Coast and Geodetic Survey Report, 1893, Part II., Appendix 3.

The camera tripod as ordinarily constructed is too unstable for purposes of topographic surveying, and it is desirable to have a tripod constructed especially for this class of work. Glass plates are heavy and awkward to carry aside from their fragile nature. Cut films can be procured in any of the standard sizes, and as they are light and stand rough handling and give ordinarily as good results as the glass plates, they are to be preferred. Their cost is about double that of the glass.

TRIANGULATION.

This method of surveying is sometimes called "Trigonometric Surveying" and sometimes "Geodetic Surveying", though this latter is properly applied only when the area to be surveyed is so extensive that allowance must be made for the curvature of the earth. Since this instruction paper is devoted to Plane Surveying only, the curvature of the earth will be neglected.

Triangulation, or Triangular Surveying, is founded upon the method of determining the position of a point at the apex of a triangle of which the base and two angles are measured. Thus in Fig. 129 the length of the base line AB is measured and the angles PAB and PBA are measured, from which can be calculated the lengths of the sides PA and PB. This calculated length of PA will then be taken as the side of a second triangle, and the angles PAC and PCA measured, from which the other sides of the triangle can be calculated. By an extension of this principle a field, farm, or a country can be surveyed by measuring a base line only, and calculating all of the other desired distances, which are made the sides of a connected series of imaginary triangles whose angles are carefully measured.

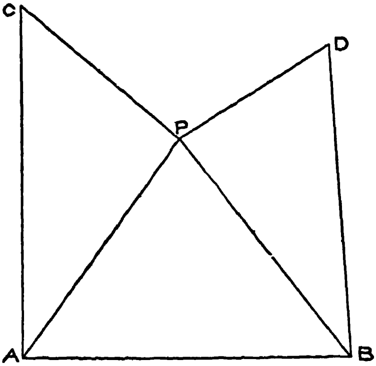


Fig. 129.

Measuring the base line. For a base line, a fairly level stretch of ground is selected, as nearly as possible in the middle of the area to be surveyed, and a line from one thousand feet to one-half mile, or longer, is very carefully measured. The ends of this line are marked with stone monuments or solid stakes. If the survey is of sufficient importance, the ends of the base line and the apexes of the triangles should be permanently preserved by means of stones not less than six inches square in cross-section and two feet long, these stones being set deep enough to be beyond the disturbing action of frost. Into the top of this stone should be leaded a copper bolt about one-half inch in diameter, the head of the bolt being marked with a cross to designate the exact point. The point may be brought to the surface by a plumb-line for use in the survey. The location of each monument should be fully described with reference to surrounding objects of a permanent character, so as to be easily recovered for future use.

The measurement of the base line for the areas of limited extent should be made with a precision of from one in five thousand to one in fifty thousand, depending upon the scale of the map, the extent of the area under survey, and the nature and importance of the work.

The two ends of the base line having been determined and marked, the transit is set over one end and a line of stakes ranged out between the two ends, especial care being taken to make the alignment as perfect as possible. These stakes should be not less than two inches square, driven firmly into the ground, preferably at even tape lengths apart, or at least at one-half or one-quarter tape lengths, center to center; the centers should be marked by fine scratches upon strips of tin or zinc tacked to the top of the stakes.

For ordinary work the base line may be measured with a tape, notes being made of the temperature, pull, grade, and distances between supports, the tape having been previously standardized. For a degree of precision, such as is attempted upon the work of the United States Coast and Geodetic Survey, more refined methods are used, but as this properly belongs to geodetic surveying, it is unnecessary to consider it here.

Measuring the angles. After establishing and measuring the base line, prominent points are chosen for triangulation points or apexes of triangles, and from the extremities of the base line angles are observed to these points, care being taken to so choose the points that the angles shall in no case be less than 30°, nor more than 120°. The distances to these and between these points are then calculated by trigonometric methods, the instrument being then placed at each of these new stations and angles observed from them to still more distant stations, the calculated lines being used as new base lines. This process is repeated and extended until the entire district included in the survey is covered with a network of "primary triangles" of as large sides as possible. One side of the last triangle should be so located that its length can be determined by direct measurement as well as by calculation; the accuracy of the work can thus be checked. Within these primary triangles secondary or smaller triangles are formed to serve as the starting points for ordinary surveys with the transit and tape, transit and stadia, plane table, etc., to fix the location of minor details. Tertiary triangles may also be formed.

When the survey is not very extensive, and extreme accuracy is not required, the ordinary methods of measuring angles may be employed. Otherwise there are two methods of measuring angles, called, respectively, the method of repetition and the method by continuous reading. When an engineer's transit is used for measuring angles, the method by repetition is the simplest and best and is carried out as follows: The vernier is preferably set at zero degrees and then by the lower motion turned upon the left-hand station; the lower motion is then clamped and the instrument turned by the upper motion upon the right-hand station: the upper motion is then clamped and the instrument turned by the lower motion upon the left-hand station; lower motion clamped and instrument again turned by upper motion upon right-hand station. This process is repeated as often as may be necessary to practically cover the entire circle of 360° and the circle is then read. This reading divided by the number of repetitions will give the value of the angle.

Now reverse the telescope and repeat the observations described above, but from *right* to *left;* the readings being taken in both directions to eliminate errors due to clamping and unclamping and personal errors due to mistakes in setting upon a station. The readings should be taken with the telescope both direct and reverse to eliminate errors of adjustments. Both verniers should be read in order to eliminate errors due to eccentricity of verniers, and the entire circle is included in the operation in order to eliminate errors due to graduation.

The second method, by continuous reading, consists in pointing the telescope at each of the stations consecutively, and reading the vernier at each pointing; the difference between the consecutive readings being the angle between the corresponding points. Thus in Fig. 130 with the instrument at zero, the telescope is first directed to A and the vernier is read; then to B, C, D, E, etc., in succession, the vernier being read at each pointing. The reading of the vernier on A, subtracted from that on B, will give the angle AOB and so on. It is necessary in this method, to read both to the right and to the left, and with the telescope both direct and inverted. Since each angle is measured on only one part of the limb, it is necessary after completing the readings once around and back, to shift the vernier to another part of the limb and repeat the readings in both directions, and with the telescope direct and inverted. This is done as many times as there are sets of readings. Each complete set of readings to right and left with

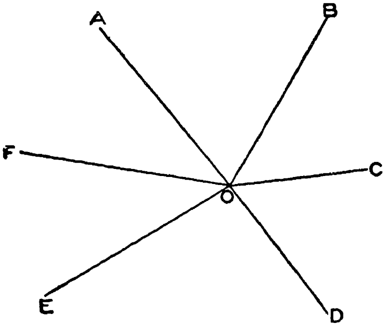


Fig. 130.

the telescope direct and inverted, gives one value for each angle.

The lengths of the sides of the triangles should be calculated with extreme accuracy in two ways if possible, and by at least two persons. Plane trigonometry may be used for even extensive surveys; for though these sides are really arcs and not straight lines the error under ordinary circumstances will be inappreciable.

Radiating Triangulation. This method as is illustrated in Fig. 131 consists in choosing a conspicuous point O, nearly in the center of the area to be surveyed. Other points as A, B, C, D, etc., are so chosen that the signal at O can be seen from all of them, and that the triangles ABO, BCO, etc., shall be as nearly equilateral as possible. Measure one side, as AB for example, and at A measure the angles OAB and OAG; at B measure the angles OBA and OBC; and so on around the polygon. The correctness of these measurements may be tested by the sum of the angles. It will seldom be the case, however, that the sum of the angles will come out just even, and the angles must then be adjusted, as will be explained later. The calculations of the lengths of the unknown sides are readily made by the usual trigonometric methods; thus in the triangle AOB, there are given one side and all of the angles of the triangle from which to calculate AO and BO. Similarly all of the triangles of the polygon may be solved, and finally the length of OA may be measured and compared with the calculated length, as found from the first triangle.

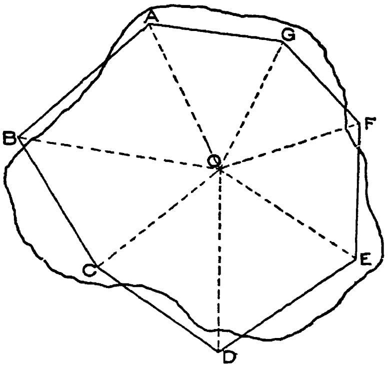


Fig. 131.

A farm or field may be surveyed by the previously described method, but the following plan will often be more convenient: Choose a base line as AB within the field and measure its length. Consider first the triangles which have AB for a base, and the corners of the field for vertices. In the triangle ACB for example (see Fig. 132), we measure the angles CAB and CBA and the length of the base line AB. We can therefore calculate the length of AC and BC. Next consider the field as made up of triangles with a common vertex A. In each of them, two sides and the included angle are given, to find the third side. If now the point B at the other end of the base line be taken for a common vertex, a check will be obtained upon the work.

A field or a farm or any inaccessible area such as a swamp, a lake, etc., may be surveyed without entering it. For a farm or any area permitting unobstructed vision, it will only be necessary to choose a base line AB, from which all of the corners of the farm, or all of the salient points of the area, can be seen. Take their bearings, or the angles between the base line and their directions. The distances from A and B to each of them can be calculated as described, and the figure will then show in what manner the content of the field is the difference between the contents of the triangles having A or B for a vertex, which lie outside of it, and those which lie partly within the field and partly outside of it. Their contents can be calculated, and their difference will be the desired content. See Fig. 133. Evidently the entire area included between the corners of the field and the base line is the sum of the triangles A2B, 2B3 and 3B4. Subtracting from this sum the areas of the triangles 2A1, 1AB, 1B6, 56B and 5B4, there will remain the required area of the field, 123456.

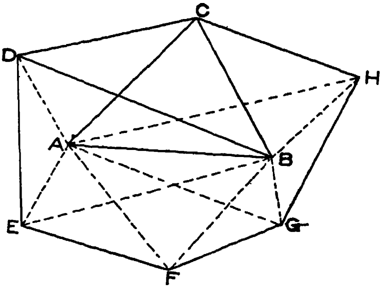


Fig. 132.

In all of the operations which have been explained, the position of a point has been determined by taking the angles, or bearings, of two lines passing from the two ends of a base line to the unknown point, but the same determination may be effected inversely by taking from the point the bearings by compass of the two ends of the base line or any two known points. The unknown point will then be fixed by plotting from the two known points, the opposite bearings, for it will be at the intersection of the lines thus determined.

The determination of a point by the method founded on the intersection of lines, has the serious defect that the point sighted to will be very indefinitely determined if the lines which fix it meet at a very acute or a very obtuse angle, which the relative position of the points observed from and to often render unavoidable. Intersections at right angles should therefore be sought for, so far as other considerations will permit.

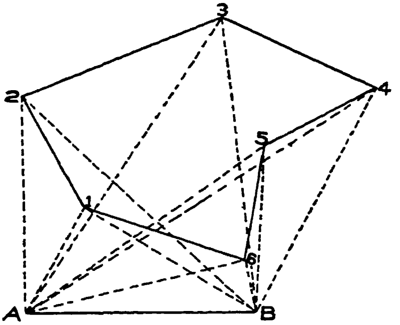
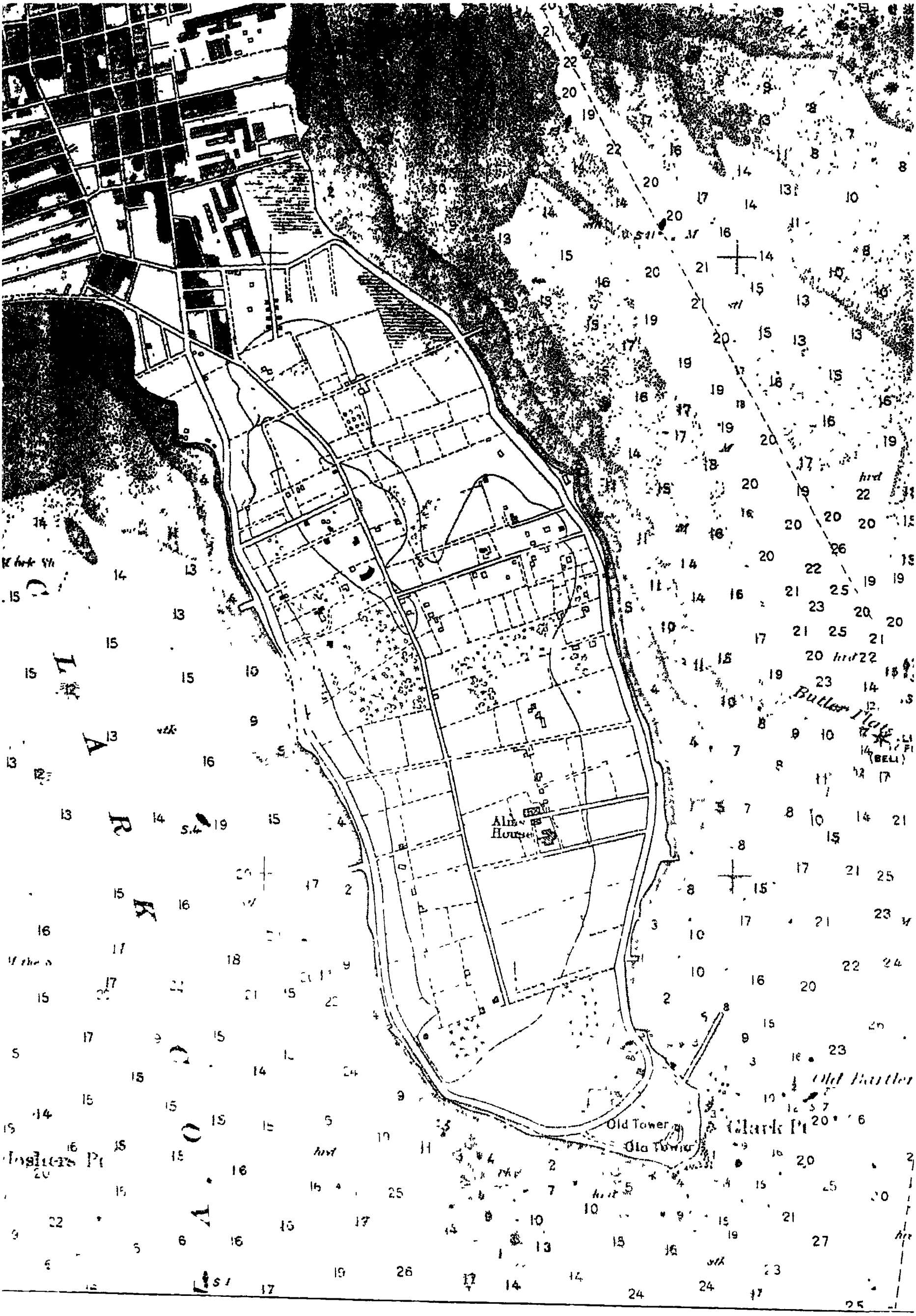


Fig. 133.

Adjusting the Triangle. All of the angles of a given triangle are measured. If but two have been measured, and the third computed, the entire error of measurement of the two angles will be thrown into the third angle. It will be found, upon adding together the measured angles of a triangle, that the sum of the three angles is almost invariably more or less than 180°. With the engineer's transit the error should be less than one minute. If there is no reason to suppose that one angle is measured more carefully than another, this error should be divided equally among the three angles of the triangle, and the *corrected* angles are used in computing the azimuths and lengths of the sides. This distribution of the error is called "adjusting" the triangle. With the large systems of extensive geodetic surveys much more elaborate methods are employed, since a large number of triangles must be adjusted simultaneously so that they will all be geometrically consistent, not only each by itself, but one with another.



SECTION OF FINISHED HYDROGRAPHIC CHART

PLOTTING AND TOPOGRAPHY.

DEFINITIONS AND OUTLINE OF WORK.

1. **Definition.** Topography is the art of making a graphic representation, (a) of the outlines of all water courses and bodies of water, (b) of all forms of relief of the surface of the ground, and (c) of all artificial features and structures built by man. In general the art consists in representing all these features in such a way that they are most easily intelligible from the map; while economy requires that the methods employed shall be such as to make a map which shall give all the necessary information with a minimum of work in surveying and drafting.

2. **Scale of Map.** (A general discussion.) One of the first things to be decided is the scale of map to be used, for on this decision will depend the character of the work which is to be done. For example, if the map is to be plotted on a small scale it will be useless to do the surveying work with a closeness of detail which it will be impracticable to represent on such a small scale. The scale of map must depend on the use which is to be made of it, as will be discussed later. On the one hand a very small scale will mean that desired details cannot be readily shown or else can only be shown by the draftsman using the most extreme care, and on the other hand, a large scale will mean that if the survey covers a considerable area of ground the map will be inconveniently large. In brief, the true criterion for the scale of the map is that it should be drawn to the smallest scale which will properly represent the topographical features which must be accurately shown on the required map.

3. **Method of Field Work.** (Determination of its general character.) The choice in the method of field work depends on the same considerations as govern the required scale of map. On the one hand, the method adopted should have a *practicable* accuracy sufficient to properly represent all required features. The practicable accuracy of any given method is only known to those who have learned by actual experience the uncertainties involved in that method But on the other hand time and effort should not be

sacrificed in plotting minute details with painstaking accuracy when it is known by experience that such accurate work cannot have any practical utilization. When an engineer is making an "exploratory" survey through a comparatively unknown tract of country, he may be content with determining elevations with a barometer, the directions of the lines with a hand compass, the slope of lines with a hand level or clinometer and distances with a pedometer (pace-measurer) or odometer (an instrument which records the revolutions of a wheel of known circumference, which is rolled over the route traversed). These crude methods have the merit of great rapidity and cheapness and can be made with sufficient accuracy to determine whether that general route should be immediately rejected or deserves further consideration and a more accurate survey. The above, which is called a *reconnoissance,* applies chiefly to railroad work. When a topographical survey of a large area is to be made regardless of the method of utilization, such a reconnoissance survey is made chiefly for the purpose of planning the detailed work, selecting triangulation stations (if the triangulation method is to be used) and determining the method best suited for the results to be accomplished. For example, military surveys are frequently made on horseback or from a boat passing on a stream or lake. The three elements of distance, direction and elevation are determined with only a low degree of accuracy, but in the hands of trained men it is sufficient for the purpose of these surveys. To pass to the other extreme, geodetic surveys, which have as their ultimate object the determination of the form of the earth, employ the most refined methods which ingenuity can devise to eliminate sources of inaccuracy which even in ordinary surveying are considered negligible.

4. **Plotting.** (The general method to be adopted.) This, similarly, depends on the method of field work and the scale of the map. Small scale maps are used to represent a very large area of country. Usually nothing is indicated except the most important features. The outlines of the largest bodies of water, such as oceans, lakes and rivers are represented as closely as is practicable. Important cities are represented by arbitrary signs such as dots and small circles. For special purposes (such as for railroad maps) the railroad is represented in a very exaggerated way. But on the other hand, as the scale of the map grows larger, more and more of the

details can be represented, until on the larger scale maps, even the nature of the crops in individual fields will be represented by appropriate topographical signs. When the relief of the ground is to be shown, and especially if it is of importance to represent it with accuracy, the usual method is to show it by contours. Since these lines are apt to be mistaken for other lines, such as roads, rivers, etc., the device of employing different colors for the various topographical signs is employed. When one individual map is to be made, it is practically as easy to use the various colors as to employ the single color of black, and it is therefore usually wise to employ different colors for such maps. But when maps are to be printed the color method will require several impressions for each sheet, which is expensive. This feature will be discussed later. If it were practicable to take a photograph of the entire area of country to be surveyed from a balloon, which was at a sufficient height above the ground to be surveyed, it would give what might be considered in some respects an ideal representation of the area to be surveyed. But such a picture would be in many respects useless. Many small but most important features would be almost indistinguishable. The essence of proper plotting consists in representing, perhaps in an exaggerated form, those features which it is especially desired to show on the map, and omitting other details which are of no practical importance, at least for the particular purpose for which that map is made. The method of plotting contours also permits an accurate representation of differences of elevation which would be unobtainable from a balloon picture.

Since the final result of a topographic survey is the map and since the method of surveying to be used depends on the scale of the map and the nature of the physical characteristics which are to be represented on the map, the various methods of mapping will be first described.

TOPOGRAPHIC MAPPING.

5. **Large Scale Maps.** Topographic maps are frequently made as the basis on which to design constructive work, such as buildings, factories, dams, reservoirs, sewerage and waterworks systems, canals, railroads, etc. Each case has its own peculiar features with regard to the topographical details which must be

represented and the accuracy with which they must be measured and plotted, but a scale of 100 feet per inch is usually large enough, unless the details are to be given with great precision or the whole area is so small that a scale of 50 or even 10 feet per inch will not make a large map. At a scale of 100 feet per inch, every building can be drawn in its actual form and dimensions (to the nearest foot), and even the separate rails of a railroad track may be clearly indicated. At such a scale, a map 30 inches square (including the border) will show about a half-mile each way or a quarter of a square mile in area. This is sufficient for many purposes and the scale is therefore largely used. There are but few natural objects (such as are usually required to be shown on maps) which cannot be shown in their natural scale. This, however, does not apply to the conventional signs used to indicate various kinds of vegetation—when it is required to show these. An accurate photographic view of these things would be useless as well as impracticable, and a conventional sign for each, which is frequently a suggestive indication of each on an exaggerated scale, must be employed even on the largest scale maps.

6. Small Scale Maps. Even when large scale maps are separately drawn to indicate details at special places, there is a great advantage in having maps on a scale so small that a very large area may be shown on a single map of convenient size. On such maps, although the general locations of buildings, etc., are desired, their exact shape or size is comparatively of no importance. The United States Geological Survey indicates residences even on maps of the scale of approximately one mile to the inch by minute squares of solid black. They are but little over $\frac{1}{100}$ of an inch square, and, therefore, would indicate (strictly) a square building a little over 50 feet square. Even a far greater approximation as to size and shape would serve the purpose. As the area to be represented grows larger this requirement must be met by (a) increasing the size of the map until it approaches the limit of convenient size for handling, (b) by making sectional maps, which for some uses are impracticable, or (c) by decreasing the scale. When the scale is decreased the same data *may* be shown by plotting the work finer. This has been carried to an extravagant limit by the United States Coast and Geodetic Survey, whose maps are exceedingly expensive to prepare, require the finest grade of draftsmen, and which almost require a microscope to determine the details. Perhaps the most practicable method of decreasing the scale is by judiciously eliminating details which will have little or no value for the immediate purpose for which the map is made, or by modifying the conventional signs employed in such a way that they will convey sufficient information without sacrificing simplicity and clearness. As we decrease the amount of matter placed on a map, we increase the clearness and ease of distinguishing those things which are shown. The United States Coast Survey maps, being made for general use, are so crowded with minute details that they have a very "flat" appearance, each line being necessarily so very fine that it requires very close examination to use the map, and this renders it difficult to use the map in a "broad" way, with the eye at a distance from it. While the inexperienced engineer should be very cautious to avoid the omission of details which might prove useful or essential and should err on the side of surveying and recording too much *in the field*, yet the experienced engineer will save time in the field and improve the character of his map by omitting details which his experience tells him will certainly not be needed in the work for which the map is to be used. When an engineer is making a map for a special purpose, such as railroad work, he has no justification in crowding the map with details which might be of importance in some other kind of work, but which will not under any circumstances be of importance in the contemplated railroad construction.

7. **Contours.** A contour is the intersection of the irregular surface of the earth by a level surface. The student should note that, although a level surface is practically a plane surface when small areas are considered, yet it is really a curved surface, being the surface that the ocean would assume if it were to rise to the assumed level. The simplest method of appreciating the nature and location of contours is to consider them as the shore lines formed by the ocean if its surface were to be raised successively to the different levels assumed for each contour. The various levels are always taken at equal vertical intervals, the intervals varying with the scale of the map and the character of the country. For the closest of detail work on a larger scale, such as might be used for landscape gardening, a contour interval of one or two feet might be necessary. But for ordi-

topography required for preliminary railroad surveys is usually done with a 5-foot contour interval. The topographical surveys as made by the United States Geological Survey, and which are plotted at a scale of one or two miles per inch (approximately), are made with a contour interval varying from 20 to 100 feet, the 100-foot interval being necessary where steep mountain ranges are to be represented.

There are certain principles regarding contours which the student must keep in mind and by which he may avoid errors in doing this work. Barring a few exceptional cases which will be mentioned later the following principles may be laid down: First, a contour is always a continuous line inclosing an area; second, since that area may, and frequently does, run off the map the contour may run from one edge of the map to any other point on the edge; third, a contour line is always a continuous line—it never stops abruptly; fourth, contour lines do not cross each other or merge into each other. The exceptions to the above rules occur only where the contours run into an artificial vertical wall or a precipice which is actually vertical. In the very unusual case of an overhanging cliff the contours might actually cross each other very slightly, but with such exceptions which would be readily recognized when they occur, the above principles may be considered as true, and when the principles are violated on a map, it may be considered as evidence of incorrect work. Neatness of work requires that the contours should be inked in by lines of uniform color and width. In order to make sure that the lines are of uniform width they should be inked in with a "ruling pen" and never with a "nib pen." If the inking in of the contours is done in one continuous operation, the lines will probably have a more uniform color and thickness. The contour elevations are always given with reference to some datum plane, such as sea level, and the elevations are always a multiple of the contour interval. For example, if the contour interval is 20 feet and the datum plane is sea level, then each contour elevation will be a multiple of twenty, even though the whole tract is far above sea level. The contours might be at elevations 660, 680, 700, 740, etc. Then the 600, 700, 800-foot contours would be made extra heavy, and could be more readily distinguished even in steep places where the contours would be very close together.

The contours should be numbered at frequent intervals, so that the elevation of a contour at any point may be determined by following it for a very short distance to the nearest "ladder" of numbers. The student is referred to Plates I, II and III for illustrations of this point as well as many others.

Hachures. The representation of the relief of a country by hachures has been almost entirely superseded by the above described method of contours. Hachures are objectionable, first, because their representation with even a pretence of accuracy and neatness is exceedingly laborious; secondly, because at their very best they do not have the accuracy of contours; and thirdly, the map is so covered by them that other desired topographical forms are practically crowded out. The chief use of such a method is to give a general idea of the character of the country when the survey has not been made in sufficient detail to render possible an accurate representation of contours. Under such conditions there is usually such a lack of detail on the map that the hachures will not crowd out the other desired topographical features. Incidentally the hachures when well done will make a very "pretty" map, when the map is really lacking in topographical detail. Maps of exploratory surveys are apt to have this character. The hachures can be used to give a very approximate representation of mountain peaks, gorges, etc., whose forms are determined only by very approximate sketching when the observer is possibly one or two miles away from them, and absolutely no instrumental work is resorted to in determining their elevation. In order to construct hachures with an approximation of accuracy it is desirable to sketch in contour lines with a pencil at equal vertical intervals. The hachures are lines which are assumed to be perpendicular to a contour line at every point of their length, or in other words, and speaking geometrically, they represent in each case and at every point of their length the steepest line at that point of the surface. On this principle the lines should be perpendicular to the contour lines and also perpendicular to any intermediate contour which might be interpolated between the contours which are drawn. This means that the hachures should be in general more or less curved. (The student should study Fig. 1 while reading this explanation.) The steepness of the slope will therefore be indicated

indicated still more graphically by increasing the thickness of the hachures on the steeper slopes. In other words, hachures are made shorter and wider for steep slopes and longer and narrower for flatter slopes. The method of hachuring has been developed so as to attempt to indicate the degree of slope by a systematic variation of the distance between the lines, but this method is exceedingly laborious and very inaccurate unless an amount of time is spent on it, which is very wasteful and impracticable. The general principle in hachure representation is that the darker the shading the steeper is the slope, and that any considerable space in white means a surface which is practically level. In Plate IV is shown a representation of the hachure method as employed on a large scale by the United States Coast Survey. The student may at once observe the exceedingly laborious character of the work and the fact that the hachures cover up a space which might otherwise be utilized in the representation of other topographical features. Hachures have the advantage of calling attention to the steepness of slopes, which may be of extreme importance on military maps where accuracy is by no means essential, but on which the fact of a steep slope, even if it is no more than a bank of earth, is of importance. Hachures may thus be used to represent an artificial or natural embankment when there is no other representation on the map showing any relief features of the ground.

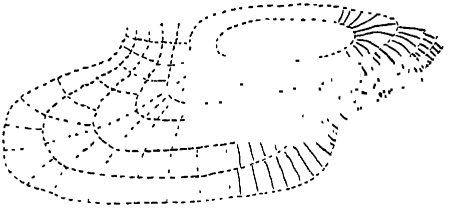
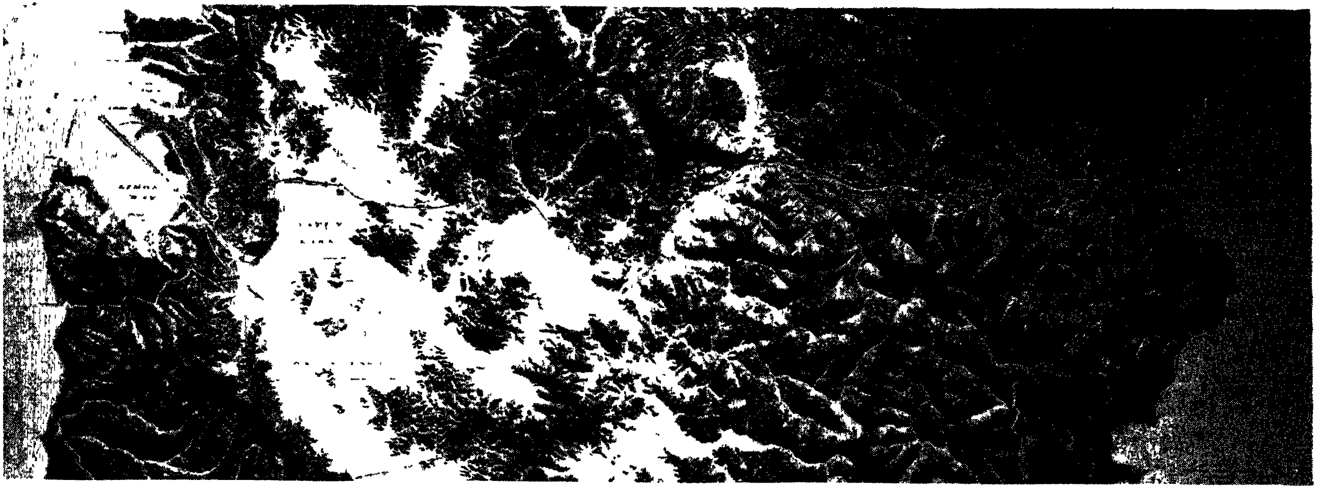


Fig. 1 Hachures (Dotted Lines Indicate Pencil Construction Lines which are Eventually Erased.)

Plate V illustrates the Coast Survey method of representing relief by means of contours and yet employ but one color. A comparison of Plate V with Plate III shows at once the great value of using three colors to represent the various topographical features. It also shows the practical difficulty even by the competent draftsmen employed by the Coast Survey to make such a completed map in but one color. Although the contours are undoubtedly more accurate and will give a close approximation of the elevation of any desired point, yet the hachured map of Plate IV is actually the more pleasing



RELIEF MAP OF PANAMA CANAL ZONE

Photograph by Howell's Microcosm, Washington, D. C.

8. Determination of Contours. The method of det tion of contours depends on the scale of the map and of the with which the work must be done. Several methods ar cribed which vary in accuracy, in the instruments emplo the labor required for doing the work.

"Gridiron" method. This simple method consists in establishing a "gridiron" of squares or rectangles over the desired area and determining the elevation of each intersection point. The great advantage of this method is its simplicity. The disadvantages are that neither the contours nor the salient points will in general agree with the points determined. Many points are observed needlessly while other essential points located within the rectangles are only determined by approximation from adjacent points. The work may be done approximately with hand level, rod and tape, but a transit and wye-level are also used with this method.

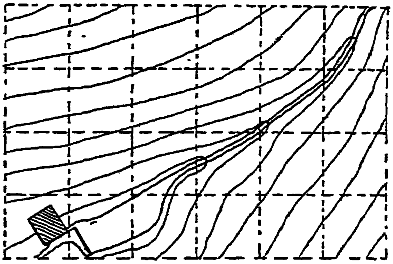


Fig. 2. Contours—Gridiron Method.

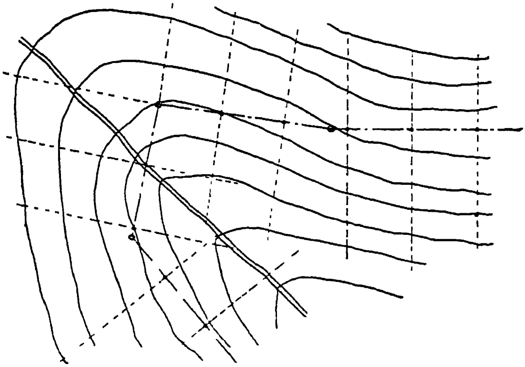


Fig. 3. Contours—Cross-Section Method.

"Cross-section" method. A broken line, consisting of straight lines between instrument stations is run with a transit or other angle measurer and a tape. At certain intervals, usually 100 feet, cross lines are run at right angles to the traverse lines. See Fig. 3. The elevation of the 100-foot sub-stations are all determined and from

them the elevation and position of points on the offset lines become known. Usually the intersection of the offset lines with the various contours is determined. The method of doing this is illustrated in Figs. 4 and 5. Knowing the elevation of the sub-stations, a point on the offset line is readily found by trial which is at the required contour elevation. The location of points with vertical intervals of 5 feet is then very simple, as shown in the figure. The contours are thus determined directly. The above methods are only used when the areas are comparatively small (but perhaps of indefinite length, as in railroad work), or when the contour interval is small and close results are essential.

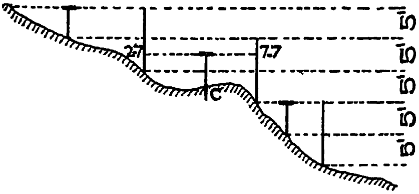


Fig. 4. Locating Contours.

"Stadia" method. This method has a skeleton like the traverse method, but even the distances between stations and their elevations are obtained by the stadia system. No 100-foot sub-stations are located. From each station point, "shots" are taken to all desired points, not only for the sake of establishing contours but also to locate all desired topographical features. Incidentally the elevations of these points, which would be determined, will assist in determining the contours more closely. It is generally found in practice that when all desired details have been located in position and elevation, the contours are determined with but few, if any, additions of points which are located solely for contour purposes. The mathematical theory of the stadia has already been given in Plane Surveying, and therefore will not be repeated here. Only practice will teach the engineer how to locate the stadia points so as to obtain all the necessary information and also to obtain the most information with the least surveying.

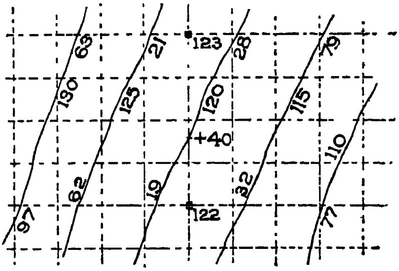


Fig 5 Contours—in Note Book.

9. **Plotting the Contours.** As before stated, the points are supposed to be so located that straight lines joining any two adjacent

points will lie approximately in the surface of the ground. On this assumption the location of any contour which comes between two located points is a mere matter of dividing the horizontal distance between the points into parts which are proportional to the differences of elevation of the given points and the contour. For example, the elevation of one point is 162.0 and of another 167.8. The 165-foot contour will evidently lie between them and at $\frac{165.0-162.0}{167.8-162.0}=\frac{3.0}{5.8}$ of the horizontal distance between them from the lower point. This is the theoretically exact solution. But since it depends on an approximation the experienced draftsman can make a rapid mental solution of the proportion which will be as accurate as the data justifies. If there are two or more intermediate contours the same general method may be applied for each, but the simplest method will be to locate first the contours immediately adjacent to each point and then interpolate any intermediate contours. After determining where each contour will cross the line joining each pair of surveyed points the contours may be drawn in by connecting the corresponding intersections. Practical modifications of this theoretical method will be made by the topographer or draftsman to allow for minute variations in the contours which may be sketched in with sufficient accuracy since the area between determined points will always be small.

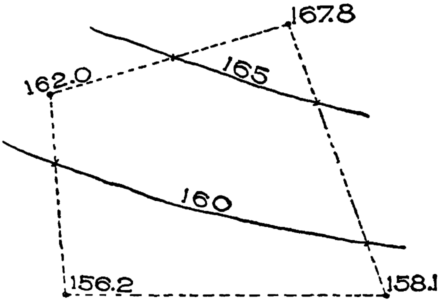


Fig. 6. Plotting Contours

Practice among topographers varies very greatly in the details of obtaining contours, varying from the one extreme of making the method as purely mechanical as possible (which implies excessive work if it is at all accurate) to the other extreme in which instruments are only used to obtain a few points of control and then to fill in the intermediate spaces by sketching. Men of long experience will do marvellously close work with very few and very simple instruing with close instrumental accuracy all of the important points and contour points with close frequency, only depending on sketching to fill in very small areas in which inaccuracies would be unable to accumulate.

10. Telemetry. This name is applied to the various methods of measuring distances without the direct use of a tape or chain. No space will here be given to a description of the many ingenious but essentially impracticable special instruments which have been devised for this purpose. In general they are incapable, for practical reasons, of giving accurate results, or else they are expensive to construct, heavy to carry, comparatively useless for other purposes, and therefore not generally adopted. The three methods hereinafter described have the advantages that they are merely attachments to an engineer's transit and that the additions to the weight and cost of the instrument are comparatively insignificant.

11. Vertical Arc Method. From one standpoint, this method hardly deserves to be classed as a regular method. Its disadvantages are so great that no one should depend on it as a standard method, but it is frequently useful when no other method is available. The essential principle is that of the solution of a triangle, of which one side and two angles are known. The rod is supposed to be held vertically. In the special and most simple case shown in Fig. 7, a rod reading is taken with the line of sight level. Then a rod reading is taken with the line of sight inclined at some measured angle. In the right-angle triangle thus formed the horizontal distance equals the rod reading divided by the tangent of the vertical angle. But in general the rod will be at some elevation with respect to the transit so that neither line of sight is level. In Fig. 8 the angles a and b are both measured. The angle $c = 90° - a$. Then in the triangle $I\ J\ H$ two angles and a side are known and the two other sides may be computed, and from them and the vertical angles the true horizontal distance and the difference of elevation may be computed. The one advantage of this method is that it can be used with any transit, which is provided with a vertical arc. The disadvantages are many; (a) two vertical angles must be measured and recorded for each point

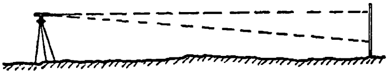


Fig. 7. Vertical Arc Method

observed; this adds to the field work; (b) an oblique triangle and then a right triangle (or two right angle triangles, as illustrated) must be solved for each point observed; this makes the office work very laborious, and it is impracticable as a regular method, in spite of the fact that it may be systematized by tabulating the results for even values of the vertical angle, the most convenient even values being used and the corresponding rod readings observed; (c) the practical accuracy obtainable is vitiated by the fact that the transit is handled and the telescope is moved in the interval of taking the two readings; if the base of the instrument were mathematically immovable during the complete operation (as it is assumed to be) there would be no trouble, but it is practically known that a microscopically minute jarring may take place which may be enough to vitiate the accuracy of the work done; (d) at a distance of 500 feet, and a rod reading intercept of 5 feet, the vertical angle would be 0° 34′ (to the nearest minute); the vertical arcs of most transits will not read closer than minutes and many of them are not dependable even for that; in the above case, an uncertainty of one-half minute in the vertical angle would mean an uncertainty of over seven feet in the distance, which demonstrates the practical limit of accuracy of the method.

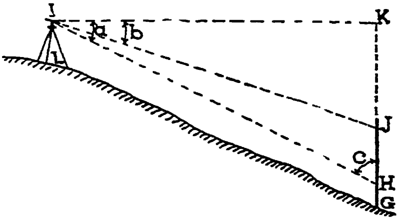


Fig. 8. Vertical Arc Method—Sloping Ground.

Example. In Fig. 8 let the angle $a = 5° 30'$, the rod reading being 1.72, and the angle $b = 4° 50'$, the rod reading being 6.45.

From the figure we may write

$$I\,K = \frac{K\,J}{\tan 4° 50'} = \frac{K\,J + J\,H}{\tan 5° 30'}$$

$$\therefore K\,J \tan 5° 30' = K\,J \tan 4° 50' + J\,H \tan 4° 50'$$

$$K\,J\,(\tan 5° 30' - \tan 4° 50') = J\,H\,\tan 4° 50'$$

$$K\,J = \frac{J\,H\,\tan 4° 50'}{(\tan 5° 30' - \tan 4° 50')} \qquad J\,H = 6.45 - 1.72$$

$$\therefore I\,K = \frac{K\,J}{\tan 4° 50'} = \frac{6.45 - 1.72}{(\tan 5° 30' - \tan 4° 50')} = \frac{4.73}{.01173} = 403 \text{ feet}$$

$K\,J = 403 \times \tan 4° 50' = 34.08$. $K\,G = 34.08 + 6.45 = 40.53$.

If the height of the instrument, $I L, = 4.90$, then the difference of elevation of L and G is $40.53 - 4.90 = 35.63$. As an illustration of the probable inaccuracy of this method, a variation of one-half minute in the value of either of the vertical angles will make a difference of over five feet in the computed value of $I K$.

12. Gradienter Method. The instruments for gradienter work are shown in Figs. 70, 71 and 75, in Part II. of Plane Surveying. The fundamental principle and use of the gradienter is described in the first part of Part III of Plane Surveying, and it therefore will not be repeated here. The method of using the gradienter to obtain distances and differences of elevation when the line of sight is inclined, are also given, and it is shown that the reduction equations are exactly the same as those required for stadia work. The practical difference between gradienter and stadia work, therefore, lies in the relative rapidity of the mechanical work and in the relative accuracy obtainable. It should be noted that the gradienter requires two settings of the horizontal cross-wire on the rod. This is sometimes done by setting the screw of the gradienter at certain even divisions, so that it is turned, say 2, 3 or 4 whole revolutions between the two sights and then reading the rod at whatever reading there may happen to be; or the two readings on the rod are taken at even-foot divisions on which the cross-wire is more easily set, and then the screw readings are read at whatever point they may happen to be. The latter method is probably the more accurate and rapid, especially if two targets placed on a level rod have been used to sight at. For all inclined sights the vertical angle must also be read. The time required is greater than that for stadia work, but the worst feature of the gradienter method is the limitation of practical accuracy. The theory presupposes that the base of the instrument is absolutely rigid during the complete operation of taking the upper and lower readings, but the telescope must be moved, the instrument must be touched, and the gradienter screw turned between the taking of the two readings. The mere fact of touching it and the interval of time required between the two readings is often sufficient to cause a motion of the instrument, which, although microscopically minute, will be sufficient to vitiate the accuracy of the work done, and unfortunately there is no practical means of detecting whether any such inaccuracy has actually occurred. In stadia work no motion of the instrument is made between taking the upper and lower readings. The instrument may (and should) be absolutely untouched. The eye can examine the reading for both upper and lower wire repeatedly to see whether any motion has occurred in the very short interval of reading the separate wires. In the case of the gradienter, the motion may or may not take place. Even if no inaccuracy were caused in this way, even once in a hundred times, there is always the uncertainty and impossibility of detecting such an error unless the readings are repeated, which, of course, would be a great waste of time. The method of reduction of stadia observations by means of tables, diagrams, or logarithmic slide rule, which is elaborated further on, may be applied equally well to the reduction of gradienter observations.

The disadvantages of the gradienter may be summed up as follows: (a) The screw thread wears rapidly until it becomes inaccurate; (b) it requires considerable time to take each observation as it should be taken; (c) the possible jarring of the instrument while any one set of observations are being taken makes the results uncertain if not actually inaccurate. The chief advantage lies in the facility with which lines of any desired grade may be run. After being properly levelled, the number of turns and fraction of a turn indicates immediately the grade of the line of collimation of the telescope in *per cent.*

13. Stadia Method. The theory of the stadia has already been given in Plane Surveying (pp. 120–125), and therefore, will not be repeated here. The methods of making and recording the field observations were also elaborated. The work of "reducing" the observations is, however, very laborious, unless it is facilitated by some of the methods hereafter described. In general every observation is made at some angle above or below the horizontal. Theoretically this reduces the nominal horizontal distance somewhat and requires a computation for the difference of elevation. Practically, the correction for the horizontal distance is too small for notice when the vertical angle is very small, but even the limitations of that correction must be known by computation. Restating the equations given on page 125 of Plane Surveying we have

$$\text{Hor. Dist. } C'N = \frac{f}{i} s' \cos^2 I + (f + c) \cos I, \qquad \text{and}$$

$$\text{Diff. Elev. } BN = \tfrac{1}{2} \frac{f}{i} s' \sin 2I + (f + c) \sin I.$$

In these equations s' is the rod intercept, f the focal length, i the distance between the stadia wires in the telescope and I the angle of inclination of the line of sight. $\frac{f}{i}$ is usually made 100.

14. Numerical Accuracy Obtainable in Stadia Measurements. Of course, a vertical arc is an essential feature of a transit used for stadia work, but the accuracy of the vertical angles depends on the accuracy with which the whole instrument is levelled. The plate bubbles of an ordinary engineer's transit, while amply accurate for their primary use of making the plates so nearly horizontal that the accuracy of horizontal angles is not vitiated, seldom have a sufficient accuracy to obtain vertical angles as close as the nearest minute of arc, and frequently their accuracy is far less than this. One method of obviating this inaccuracy is to determine by repeated reversions of the instrument, while it is being set up, that the horizontal plate is truly horizontal, or in other words, that the vertical axis is truly vertical, thus eliminating any error of adjustment of the plate bubbles. But even this method, unless repeated every few minutes at a costly sacrifice of time, will not prevent inaccuracies due to the settling of the instrument; nor will it at its best render possible any close accuracy of the vertical angles, since the bubbles are not sufficiently sensitive. A transit for stadia work usually has a large bubble underneath the telescope, which is much more sensitive, and on which a movement of 30″ of arc will correspond to about one division. Even if this bubble is in perfect adjustment, it will be necessary to make a test every few moments by clamping the telescope at exact horizontality and noting whether the vernier of the vertical arc reads zero or reads the amount of its "index error." When it does not do so, it indicates that the instrument has settled somewhat in the ground and has thereby become thrown out of level to some extent. A far better method is to have the vernier of the vertical arc readily adjustable and with a sufficiently sensitive bubble attached to it. When this bubble has been properly adjusted, any slight motion of the instrument due to settlement can be readily neutralized by an adjustment of the vernier arc, which will then give each vertical angle correctly. But it should be noted that since vertical angles are only read to single minutes, the sights should not be made excessively long except when they are taken merely for somewhat approximate purposes in which a considerable inaccuracy would be tolerable. For example, at a distance of 1000 feet a difference in vertical angles of 0° 1′ will equal .29 of a foot or 3½ inches. Such a stadia shot would be justifiable at that distance to obtain a "contour point," but unless the whole work was approximate, a stadia station should not be located with such inaccuracy. On the other hand, for short distances the stadia method gives results which are amply accurate for its purpose. It is not, of course, supposed that the method would be used to replace wye leveling. But when, as is usually the case, the stadia method is used for preliminary surveys or else to fill in the topography between the points whose relative positions and elevations have already been determined by triangulation and by leveling, then errors have no chance to accumulate and the method has the combined advantages of great rapidity, cheapness and sufficient accuracy for the purpose. One great bar to its use has been a lack of knowledge regarding the principles involved or a lack of knowledge of the fact that the reductions can be so readily made by means of slide rules. Much of the work now being done is "reduced" by the use of tables and numerical multiplications, which are certainly very slow and laborious and which would almost condemn the method for practical use if they could not be avoided. In making the above reductions, the student can solve them by the use of trigonometrical tables, although that is the most laborious method. A set of stadia tables (see Table I) is given with this course of instruction, and the student should solve the problems given later on, using these tables; if in addition he can secure the use of a stadia slide rule, he can determine for himself the economy of time obtainable. Of course the student must expect that it will be a little difficult at first to use any stadia slide rule until he becomes accustomed to its manipulation, but a short experience with it will convince him that it will pay for itself in a very few days. If possible, the student should solve the problems (as well as others which he can himself make up) by all of the three methods referred to, and thus determine for himself the relative facility of doing the work

OFFICE WORK.

15. Reduction of Observations. The ordinary numerical trigonometrical reduction of each observation by the equations of § 13 is so laborious that such a practice would render the method impracticable; $(f + c)$ is usually about 12 to 15 inches; $\cos I$ is always less than unity, but for ordinary small angles is but little less than one; therefore $(f + c) \cos I$ is always very nearly one foot. Considering that one foot is the smallest unit of measurement which it is practicable to use in observing stadia *distances*, the practical accuracy is not vitiated by considering that term invariably as one foot. When the angle I is about 5° 45′, $\cos^2 I = .99$ and the horizontal distance would be 1 per cent less than its nominal value of $100\, s'$, or 100 times the rod reading. At one-half this angle (2° 52′) the correction would be about $\frac{1}{4}$ of 1 per cent. This gives an indication of when the correction will be large enough to deserve accurate computation. It may be also observed that for small angles $\sin I$ is nearly equal to $\frac{1}{2} \sin 2I$; for 10° the discrepancy is about $\frac{1}{4}$ of 1 per cent; therefore we may say approximately that the difference of elevation $BN = (100\, s' + 1)\ (\frac{1}{2} \sin 2I)$. For angles of less than 10°, such as would usually be the case between stadia stations when the levels are particularly important, the error is a very small fraction of an inch and unobservable with the instrument. Even with a vertical angle of 30°, the error of the approximation would be less than 0.1 foot, which is the lowest unit for contour levels. We may therefore deduce the following practical method in which all approximations are within the lowest units of measurements. For small vertical angles, the horizontal distance equals 100 times the rod reading plus one foot. For larger vertical angles multiply 100 times the rod reading by the square of the cosine of the angle of inclination and add one foot. The practical method of doing this will be given later. For the difference of elevation, add one foot to 100 times the rod reading and multiply this by one-half the sine of twice the angle. There are three shortened methods of accomplishing these trigonometrical multiplications; (*a*) *tables*, (*b*) *diagrams*, and (*c*) *slide rules*.

The first is by means of tables which shorten the operation to the extent of giving directly $\cos^2 I$ and $\frac{1}{2} \sin 2I$ for all desired values of I, but this involves two numerical multiplications which are tedious. See Table I.

The second method is by means of a large chart or diagram. There are many forms of these, but they are usually essentially as follows: there are two sets of ordinates, one representing distance (the nominal rod reading times 100) and the other the desired quantity. Lines are drawn from the origin for each desired angle so that the intersection of any angle lines with any distance ordinate will give an ordinate of the other system which will give the desired quantity. The practical difficulty lies in the fact that if made conveniently small, the scale is so small that the results are inaccurate. When they are sufficiently enlarged they are clumsy to handle. The larger sizes should be used and they are capable of good work.

The third method is by means of slide rules. These are but an adaptation of the ordinary slide rule since the process is merely one of multiplication. One scale is marked with degree and minute marks at the proper places for the numerical values of those trigonometrical functions. By setting the zero of the angle scale at the given distance (100 times the rod reading) on the other scale and then noting the mark for the observed vertical angle, it will be found opposite the required difference of elevation. The true horizontal distance may be most easily found by modifying the above formula so as to compute the *correction* to the horizontal. $\text{Cos}^2 = 1 - \sin^2$. Therefore by multiplying 100 times the rod reading by $\sin^2 I$, we obtain a correction, which is seldom more than 5 or 10 feet, which is subtracted from the nominal distance. In the practical use of the scale it will be found that this quantity may be more readily read off from the scale than the true horizontal distance, and as it is only a few feet the subtraction is readily made mentally while it is being recorded. Colby's slide rule only gives the difference of elevation. The K. & E. stadia slide rules are engine-divided on white facings and give both the difference of elevation and true horizontal distance with one setting of the rule, but the length of the logarithmic unit is very short, which makes it correspondingly difficult to obtain close values. The Webb stadia slide rule has a logarithmic unit 12.5 inches long, which was made possible by the use of the cylindrical principle, and this enables the values to be easily read with as close accuracy as stadia work will justify. The horizontal distance is obtained by computing the *correction* to the horizontal from the $\sin^2$ function

16. Numerical Examples. 1. The rod reading for a stadia shot was 3.74 and the vertical angle was $+ 6° 22'$. What is the true horizontal distance and the difference of elevation? The horizontal distance would be, according to the approximate rule, $100\, s' \cos^2 I$. Cos $6° 22' = .9938$. Squaring this number we have .98765, which, multiplied by $374 = 369.4$. Adding 1 foot we have 370.4. The only approximation in the above is on the assumption that $(f + c) \cos I = 1$ foot, but as before stated, the error of the approximation is so small that it may be disregarded. In fact, the tenths in the distance given above should also be disregarded, since the rod reading is taken only to the nearest hundredth of a foot, which corresponds to an even foot in horizontal distance. The difference of elevation is evidently equal to $\frac{1}{2} \times 100 \times 3.74 \times \sin (2 \times 6° 22') + (f + c) \sin 6° 22'$. This reduces to $41.22 + 0.11 = 41.33$. Since $\sin 6° 22' = .1109$ while $\frac{1}{2} \sin 12° 44' = .1102$, the approximation referred to above only equals $.0007\, (f + c)$ which equals in this case .0084 inches when the $(f + c) = 12$ inches. Since it is useless to attempt to take levels closer than 0.01 of a foot which equals .12 of an inch it is thus seen that the above approximation is far within the limits of accuracy. Therefore we can see at once that the difference of elevation equals $(374 + 1)\, \frac{1}{2} \sin 2I$, which in this case equals 41.33, which is the same value as before.

The above solution was made by the long and usually impracticable process of taking the multipliers from an ordinary trigonometrical table. The method is simplified by employing what is called a "stadia reduction table," such as is given in the back of this book. Looking in the column headed 6° we find opposite 6° 22′ the value 98.77. If we multiply 98.77 by 3.74, the rod reading, we obtain 369.4 as before; adding 1 we have 370.4. For the difference of elevation we multiply the number corresponding to 6° 22′, which is 11.02, by 3.74, and obtain 41.21. This practically is the same value as before, the difference, which is really but a fraction of a hundredth of a foot, being caused by a slight inaccuracy in the last figure in the table. At the bottom of the table we find that when $(f + c) = 1$, as we have assumed above, the quantity to be added for difference of elevation = .11, which gives a total of 41.32, which is the value we previously obtained by the long process. This process is therefore considerably shorter than the first, although it involves two multiplications. The third method is to use a diagram. As it is impracticable to illustrate a diagram which would have any practical value for this purpose on the pages of such a book, we must be content with a mere description of the process as described in section 15. Stadia slide rules are illustrated later on. (See Section 33, Figs. 25, 26 and 27.) To use them we simply need to set the zero of the angle scale (or perhaps some other specified mark in a multiple scale) opposite the mark on the distance scale which represents the rod reading; then opposite the mark on the angle scale, which indicates the angle of elevation or depression, we may read directly on the distance scale the difference of elevation. For example, in the above case, and using the Webb Stadia Slide Rule, we would set the left-hand end of the scale, which gives the "difference of elevation" and which contains the angles between 6° and 7°, on the mark indicating 375, then looking for 6° 22′ on the angle scale, which would be found by interpolating between the mark for 6° 20′ and 6° 25″, we may read at once 41.3. This is always sufficiently accurate for contour work, and unless extreme care has been taken to have the instrument in perfect adjustment it is as close as the difference of elevation can be depended upon at a distance of 375 feet. This operation requires but a few seconds of time. To obtain the correction to the horizontal distance, we place the left-hand end of the rule giving "Hor. corr." for 6° 22′ on 375 of the main scale and note that the angle marked 6° 22′ occurs at $\frac{1}{100}$ of 460, showing that the horizontal correction is 4.6, which would give the true horizontal distance as 370.4 as before. This is even more accurate than is necessary, and we would call the true horizontal distance the even number 370 feet.

Example 2. Verify the following results obtained with an instrument whose $(f + c) = 12$ inches. It should be noted that a large vertical angle is not usually combined with a great distance, as that would mean an extreme difference of elevation. The following examples have purposely been chosen to cover almost every case that will occur in actual practice; namely, short distances and slight difference of elevation; long distances with small vertical angle; and short distances and large vertical angles, such as occur when surveying through a gorge and side shots are taken up the steep banks of the gorge. The student should note in each case that the approximate

Rod Reading.	Vert. Angle.	Diff. Elev.	True Hor. Dist.
1.34	+ 0° 14′	+ .54	135′
8.76	+ 0° 12′	+ 3.06	877′
4.22	- 7° 06′	- 51.9	417′
0.96	+ 25° 15′	+ 37.47	80′

17. Reduction of Field Notes. This topic is mentioned chiefly with a view of giving a warning. With a few exceptions which will be noted, there should be no such thing as a "reduction of field notes," or in other words, the notes taken in the field should be complete and the surveyor should never wait to get back to the office and then depend on his memory to complete the notes. A desire to make the notes neat and to do the work under more favorable circumstances of office surroundings and suitable temperature, will often tempt the surveyor to make the field notes very meagre and then depend on them and on his memory to make a more complete set after the office is reached. Under special circumstances this may be justifiable, but in general every essential fact should be recorded in the field. The only exception to this may be stated in a broad way to be such reductions and additions as can be unmistakably made from the field notes taken. An example of such reduction is the computation of the difference of elevation and the correction to the horizontal distance which must be computed from stadia measurements. When circumstances will permit, it is preferable to make even these reductions in the field, as may be readily done if a stadia slide rule is used, and then plot the work on a sketch board. The surveyor should cultivate the habit and ability to make sketches in the field which shall be approximately to a correct scale and which will settle indisputably the ambiguity which often arises from field notes expressed simply in words and numbers. Therefore with the exception of mathematical processes, which are perfectly definite and which may be made more conveniently in the office after the surveying work is done, there should be no "reduction of field notes."

18. Plotting of Farm Surveys from Field Notes. Some of the statements in the last paragraph apply especially to this subject. A sketch of the farm, as is illustrated in "Plane Surveying," page 119, Fig. 83, should always be made in the note book. Such a sketch is of even greater importance than the more formal notes in columns such as are illustrated on page 118, for the sketch alone would serve the purpose if care is taken to record everything on it, while the notes in columns are frequently subject to ambiguity. To put it more strongly, it requires the greatest skill to make the formal notes so that they will not be ambiguous. The plotting of farm notes will therefore consist of making a copy of the sketch, but with all dimensions drawn accurately to the desired scale instead of roughly and inaccurately as in the sketch. The scale of sketch to be adopted of course depends on the amount of detail which it is desired to show on the map. If possible, the map should be oriented so that the meridian points up, but this may be impracticable on account of the form of the farm, or the direction of its longest dimension. As a matter of practical accuracy, the direction of the meridian line should be immediately determined by penciling a long line that runs approximately through the center of the map, then all lines should be plotted in accordance with their angle from this meridian line. If latitudes and departures have already been computed it is far more accurate to determine the direction of the boundary lines by scaling off the latitude and departure of each line. If it is necessary to use a protractor, then the protractor should be as accurate as possible. The very small protractors having a radius of about 2 inches, which are frequently found with a cheap set of drawing instruments, are worthless for accurate topographical plotting. If a protractor must be used, it is far better to purchase an 8-inch or even a 14-inch paper protractor which can be bought for 20 or 40 cents. With care this will last a long time and will be far more satisfactory than the small metal or horn protractors which are frequently used.

Another method of laying off angles accurately is to use a table of natural tangents, using a decimal scale, say 20 to the inch. Five inches of this scale is divided into 100 parts and by interpolation it may be read to 1000 parts. Lay off this 5 inches on a line and erect a perpendicular to that line at the 5-inch point. Suppose that the angle to be laid off is 16° 45′; the natural tangent taken from a table is 0.301. Therefore, scale off on the perpendicular with the "20 scale" 30.1 divisions. A line to the starting point will make the required angle of 16° 45′ with the given line. Since the length of approached 90°, the preferable method would be to erect a perpendicular to the given line at the point from which the angle was to be made; lay off the 5 inches on that perpendicular, erect a line perpendicular to it (or parallel to the original line) and determine the tangent of 90° minus the given angle. Of course, if still greater accuracy is desired, a longer base line than 5 inches can be used. Other details of the plotting of farm surveys, which are applicable to other forms of plotting, will be given subsequently.

19. Plotting of Profile From Level Notes. Profiles are almost invariably plotted on "profile paper." There are three general methods of ruling the lines on profile paper, and the paper adopted will depend somewhat on the use to be made of it. Vertical lines are ruled $\frac{1}{4}$ inch apart, sometimes $\frac{1}{5}$ inch apart. Horizontal lines are ruled 20, 25 or 30 to the inch. The paper comes either in sheets about 42 inches long and 15 inches high, or else in rolls which are 10 or 20 inches wide and 50 yards long. Sometimes the paper is mounted on muslin and sometimes the rulings are made on tracing paper or on tracing cloth. The levels are always referred to some datum plane, which is lower than any elevation to be recorded. Each fifth horizontal line is ruled somewhat heavier than the others and one of these lines should be selected as the line indicating some even multiple of 5 feet above datum plane. Some little ingenuity is often necessary to choose the proper line, so that the profile will run neither too high nor too low. The scales, both horizontal and vertical, which are to be used, must be selected with reference to the character of the work to be shown. The horizontal scale will usually vary from 100 to 400 feet to the inch. At a scale of 400 feet to the inch, each vertical line therefore corresponds to 100 feet of horizontal dis-

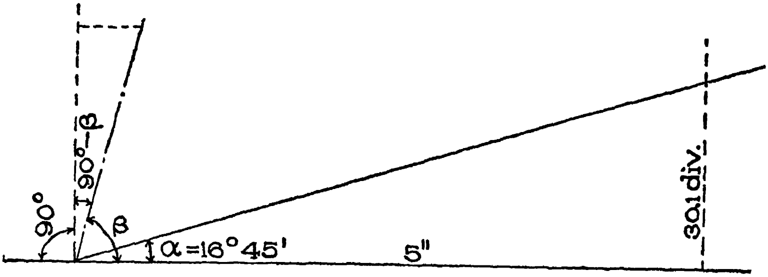


Fig. 9. Plotting Angles from Tangent.

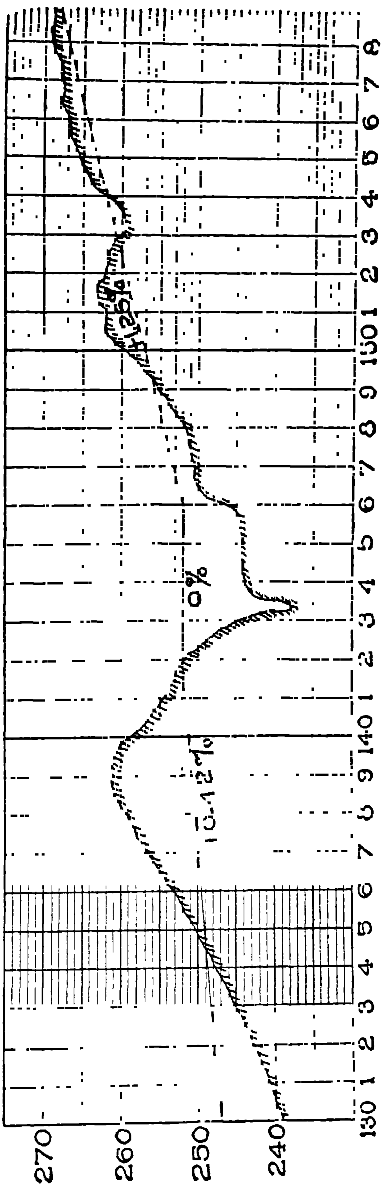


Fig. 10. Profile and Grade Line.

tance. Usually each horizontal line (spaces $\frac{1}{20}$ of an inch apart) means 1 foot of elevation, or in other words, the vertical scale is 20 feet to the inch. Since the engraved lines usually cover a vertical space of 15 inches, this scale will allow a range of profile of 300 feet. If 20 feet per inch is chosen as the vertical scale, and the total vertical range of the profile does not exceed 300 feet, then the whole profile may be placed without a break on such a sheet, but the choice of lines to represent any given elevation must be made accordingly. When a larger vertical scale is chosen or the range of elevation is greater, as it is apt to be for a continuous railroad profile, a break may be made by raising or lowering the profile an even number of feet; for example, by making a change at some vertical line by which the elevation indicated by a given horizontal line is abruptly changed, say 100 or 200 feet.

Mark on the profile sheet the elevation corresponding to the horizontal lines and also the distances corresponding with the vertical lines, then having recorded in the note-book the elevations above the datum plane of every desired point in the profile, plot each point at its proper position according to the horizontal scale and likewise according to the vertical scale. A line drawn through these several points will give the profile of the surface. If it is desired to establish a given grade line on that profile, the location in distance and elevation of the governing points of that grade line should then be plotted and the grade line drawn through them. Usually this grade line will lie alternately above and below the surface line and we thus have a graphical representation of the amount of cut or fill required at any point.

20. Plotting of Stadia Survey Notes. Stadia surveying is almost invariably based on a skeleton which has been surveyed with a higher grade of accuracy than is possible with mere stadia work. This skeleton is usually made by triangulation, and therefore errors cannot accumulate beyond those incident to a survey between two triangulation stations. Of course, the sub-stations between the triangulation stations are determined wholly by stadia measurements, and if the area to be surveyed is very small, this may constitute the skeleton of the whole work regardless of any triangulation stations. If triangulation has been used, then the triangulation stations should be first plotted with all the accuracy that is possible. The next step will be to run the traverse lines which connect the stadia sub-stations and which run between the triangulation stations, as well as shorter traverse lines which might be used to connect stadia stations and form a net work. The stadia work for the lines between stations should be done with such accuracy that there is no appreciable error or closure when connecting the triangulation stations. If any such error is found, it should be investigated, and if it amounts to a great deal, it may indicate some gross blunder in the field work. One method of investigating such blunders is to compute the latitude and departure of each course of the traverse. The algebraic sum of these should be the latitude and departure between the two terminal stations, and the amount of the discrepancy will indicate whether it should be ascribed to mere cumulative inaccuracy or to some gross blunder at some one place. When the closure has finally been settled with satisfactory accuracy all stadia stations will have been plotted. The reference line for azimuths in stadia work is usually a meridian line, but whether it is so or not, that reference line should be drawn through each station and sub-station from which stadia shots have been taken. A full circle protractor is the most convenient instrument to use for this purpose, and although a protractor 6 inches in diameter, when used carefully, may answer the purpose, it is preferable to have one of a larger size even though it is engraved on paper. Number all the stadia shots at each station and with its zero line running along the azimuth reference line, mark by dots on the edge of the protractor the angles as given for the various shots taken from that station. As each one is marked, indicate the mark very lightly by a number, which is the number of the shot. Usually all the shots taken from any one station may be marked off with a single placing of the protractor; then, removing the protractor, scale off from the stadia station the various distances for each shot, making a dot at the required distance and drawing around it a small circle very lightly in pencil to aid in finding its position. Indicate immediately in pencil the elevation of the point, which elevation should have been previously computed and recorded in the note book. This also serves to identify the point. When two or more points have been obtained to determine the corners of buildings, bridges or other structures, the points should be immediately connected so as to render the significance of the points at once more intelligible. It may even be preferable to indicate very lightly in pencil the general run of the contours through the elevations which have been obtained, since every new indication on the map will render it plainer and enable the rest of the work to be put on with less danger of making blunders. By making all marks with a hard pencil and without bearing on very hard, mistakes may be corrected without excessive erasures, which might spoil the paper, and yet the penciling will be sufficiently plain so that it can be inked in when it is found to be satisfactory. This is particularly true in the matter of connecting the contour lines which have been determined from various stadia stations. After indicating all desired features, the azimuth reference line through each stadia sub-station, as well as other features which have been lightly penciled in, may be erased without trouble.

21. Plotting of Maps by Projection. For small areas such as farms and parks, where the area is but a few square miles, it is sufficiently accurate to assume that the surface of the earth is plane, or in other words, that all plumb lines held at all points of the map would be precisely parallel. But when the areas amount to hundreds of curvature of the earth cannot be neglected either in the computations or plotting. A "single curved surface," such as that of a cone or cylinder, can be "developed" and it will lie flat; *i.e.*, the surface could be rolled out on a plane surface without changing any proportions. But a surface of "double curvature" such as a sphere is not developable, and there then arises the problem of representing on a plane surface the surface of a sphere with as little distortion as is possible. There are many methods of accomplishing this, some of which are very approximate. They will be described in the order of their complexity, which likewise means the order of their accuracy. In each case will be described the method of plotting parallels of latitude and meridians of longitude. Since in any case a line is determined by its passing through a point of known position and running at a given angle with the meridian, the plotting of meridians and parallels gives a sufficient framework for the plotting of any map.

Trapezoidal Projection. In this method parallels and meridians are plotted as straight lines. In Table II are shown the distances between parallels of latitude at any given latitude (the distance varying slightly with the latitude) and also the distance between meridians on any parallel of latitude. A straight vertical line at the center may indicate the even degree meridian which passes approximately through the center of the map to be plotted. On this line lay off distances at the proper scale which will represent the distances between the parallels of latitude included within the map. On two of these parallels, which are approximately at one-fourth the distance from the top and bottom, lay off distances from the central meridian, which represent, according to Table II, the positions of even degree meridians at so many degrees each side of the center. Connecting the corresponding points on these two parallels we have a series of trapezoids, each of which have parallel lines for top and bottom and a pair of converging lines for the sides. When the scale is such that the angular distance of any point from the center of the map is but a few degrees measured in latitude or longitude, then this method might be tolerated, although the errors at the outer edges of the map would be considerable. When the map includes many degrees of latitude and longitude, then the errors become intolerable.

Simple Conic Projection. A cone is readily developable and its development is a sector of a circle whose radius is the slant height of the cone. If we consider a cone to envelop a sphere so that it in the pole of the sphere and is tangent to the sphere at some p of latitude, which is purposely chosen as the middle parallel o desired map, then if points on the surface of the sphere are projected on to the surface of the cone and the cone is then developed, we will have a representation of the surface of the earth with comparatively little distortion, especially when the angular distance measured in degrees of latitude from the middle parallel is very small. Fig. 12 illustrates this method on a very small scale, but it also shows that when the angular range of the map is very large the distortion on the outer edges of the map is very considerable. The distance between parallels 45° and 60° is very perceptibly less than that between 0° and 15°. A very obvious modification of the method is to space the parallels at their true distance apart. To apply this method lay off as before a vertical line which represents an even degree meridian passing approximately through the center of the map. On this meridian lay off, as before, points which represent the distances between consecutive parallels of latitude. From Table II we may take the length of the slant height of such a tangent cone, which is tangent at the middle parallel of the map; spacing off distances on this central meridian we determine the position of the apex of the cone of development. We may then draw through the points on this meridian, which represent the various positions of the parallels, arcs of circles which will represent the developable parallels. On the middle parallel of the map we

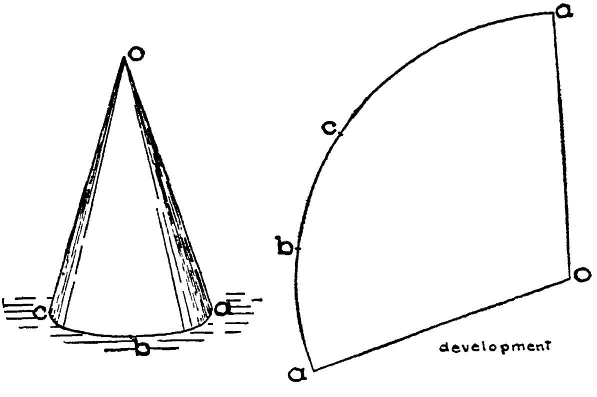


Fig 11 Development of Cone

may plot the position of the various meridians included in the map and through these points we may draw radial lines in the direction of the apex of the developable cone. Points in the immediate neighborhood of the middle parallel are correctly plotted. Points which are further away from the middle parallel, either above or below, are more and more inaccurate; but this method will answer for plotting of areas of several hundred miles. One very practical difficulty, which is almost insuperable unless the scale of the map is very small, is that the slant height of the developable cone is too large and the apex of the cone is really inaccessible. For example, at latitude 40° the length of the side of the tangent cone is 4730 miles. Even if we adopt a scale of 1 ÷ 125000, which is approximately two miles to the inch, the length of the radius to be used in developing this cone would be 4730 ÷ 2 = 2365 inches = 197 feet. It then becomes impracticable to use this method except by adopting a method of "co-ordinates" which will be described later.

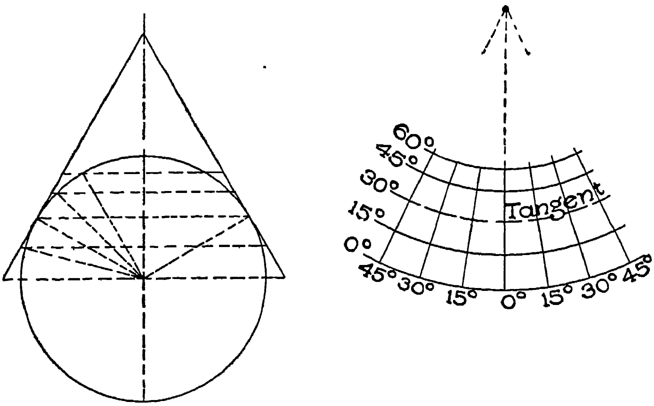


Fig. 12. Simple Conic Projections.

Simple Conic Modified. The simple conic is totally outside of the sphere, except at the circle of tangency. The map is therefore always larger than the true area. A simple modification is to lay off on two developed parallels situated at one-fourth of the height of the map from top and bottom the true distance between meridians. The meridians are then drawn as straight lines through these points of division. This is virtually the equivalent of the development of a cone which intersects the sphere between the two selected parallels, the cone being inside the sphere between the parallels and outside of the sphere above and below them. That part of the map which lies between the two parallels will be slightly too small and that above and below will be slightly too large, but in no case will the variation be as large as in the case of a simple conic. A still further modification is to lay off on each parallel the proper distance between meridians for that latitude, but the parallels are developed as arcs of concentric circles. Each of these successive methods is a little more accurate than the previous method, but unless the scale of the map is such that it is practicable to draw the developed parallels directly as arcs of circles and without the use of coordinates, it is just as simple, as well as being much more accurate, to employ the following method, which is the method used by the United States Coast and Geodetic Survey, as well as by all other geodesists.

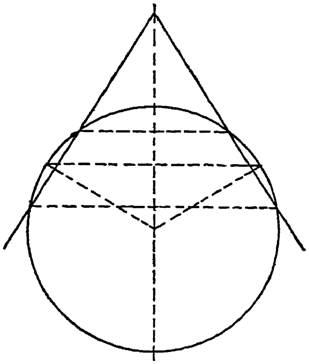


Fig 13 Simple Conic—Modified.

Polyconic Projection. Let us consider a sphere with parallels of latitude and meridians drawn over it and then assume that the sphere is covered with a series of cones whose vertices are all in the pole of the sphere, each cone being tangent to the sphere along one of the parallels of latitude. If we then develop these cones, the line of tangency of each cone will develop as the arc of a circle whose radius is the slant height of the cone, and the intersection of the tangent parallel with each meridian may be plotted on this circle in its true position. By drawing a vertical center line we may lay off on it the true distance between the desired parallels of latitude; then, laying off on that vertical center line from the position of each parallel, a line equal to the slant height of the corresponding tangent cone, we may draw arcs which will represent the development of each parallel of latitude. To illustrate the principle involved this has been done on an exceedingly small scale and by lines which represent the development of many degrees of both latitude and longitude in Fig. 14, but this method is particularly used for the plotting of maps on a comparatively large scale in which the length of the radius of the developing cones (at the scale adopted) would be impracticably large. It therefore becomes necessary to plot these curves by the method of co-ordinates, which must necessarily be computed and arranged in a tabular form for use. It then becomes just as easy to use co-ordinates which are computed with reference to the true form of the earth rather than those computed for a sphere. In Table II is given for various latitudes the length in statute miles of 1° of latitude, the length of 1° of longitude, the length of the side of the tangent cone, the distance from the center meridian of 1°, 2°, 3°, 4° and 5° of longitude, and also the vertical ordinate for each one of these points above a horizontal tangent line. The method of drawing one of these maps may be most easily described by a concrete example, as follows: *First.* Draw a vertical line to represent some even-degree meridian of longitude which is as near as may be at the center of the map. Having selected the scale, space off on this vertical line the positions for as many parallels of latitude as it is desired to represent. The length of 1° of latitude measured on a meridian for various latitudes in statute miles is given in the accompanying table. For example: suppose that the scale of the map is to be $\frac{1}{125,000}$. This is

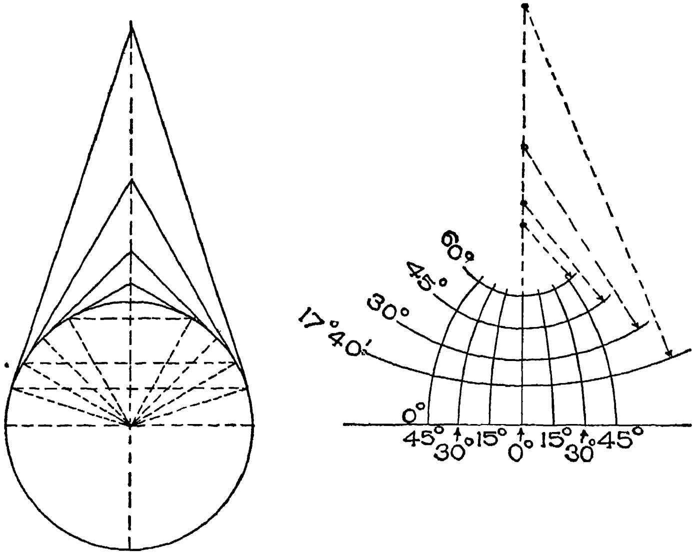


Fig. 14 Principle of Polyconic Projection

approximately equal to a scale of two miles per inch. At latitude 40° the length of 1° of latitude in statute miles, as taken from the table, is 68.993. Multiplying this by 63360 we have 4371396 inches, which is the length of 1° at 40° latitude, or which is the length between 39° 30′ and 40° 30′. Dividing this by 125000 we have 34.97 inches, which will be the distance to be scaled on the vertical line between two marks, which should represent the parallels 39° 30′ and 40° 30′. Dividing this space in half we have the position for 40° latitude and any other desired parallels may be interpolated accordingly. This is, of course, on the assumption that 40° is the mean latitude. *Second.* From each point on the center line through which a parallel is to pass, draw a horizontal line. Assume that we wish to lay off meridian lines at intervals of 0° 15′ of longitude. The accompanying table, which has been compiled from the more extensive table of the United States Coast and Geodetic Survey, published in their report for 1884, gives the co-ordinates of points which are from 0° 15′ to 3° east or west from the center meridian of the map for all latitudes between 25° and 50°. These co-ordinates are given in meters since this is the universal unit of measure in high grade geodetic work, but these co-ordinates can be readily transferred into other units on the basis that one meter = 3.2809 feet = 39.3704 inches. Although the accompanying table is amply accurate for much of the work that an engineer may have to plot, the more extensive tables of the Coast Survey should be utilized for the highest grade of work. Since 1 meter = 39.3704 inches, when we are working on a scale of 1 ÷ 125000 each meter would scale .00031496 inches or 10000 meters = 3.1496 inches. For our present purpose we may call this 3.15 inches. According to our table the distance out from the center for 0° 1. at latitude 40° is 21349 meters, which at 3.15 inches per 10000 meters gives 6.72. Similarly we can reduce to inches the ordinates for 0° 30′, 0° 45′ and 1°. The distance out for 1° 15′ can be obtained sufficiently close by interpolation between the values for 1° and for 1° 30′. At all these points we can erect short perpendiculars. This should be done on both sides of the vertical center line. The vertical ordinates which are given in the table may be similarly changed to inches. The vertical ordinate in meters for 1° from the center is 479 meters, which at .000315 inches per meter = 0.151 inches. For 2° the vertical ordinate is four times this or 1916 meters or 0.603 inches, thus indicating that the vertical ordinates increase as the square of the distance from the meridian. Therefore the ordinates for 1° 15′, 1° 30′ and 1° 45′ will = $\frac{25}{16}$, $\frac{36}{16}$ and $\frac{49}{16}$, respectively, of the unit height for 1°, which is 0.151 inches. For a width in latitude of 3°, each side of the center, other points may be similarly obtained. Having plotted these points for 40° latitude, we must repeat the operation for each new parallel required. The figures to be used for intermediate parallels of latitude may be obtained with more than sufficient accuracy by interpolating between the numbers given in the tables.

It should be noticed that Fig. 14 was drawn mainly for the purpose of illustrating the general principles involved. It covers a range of 45° each side of the center line and from the equator to 60° north latitude. This is a very extreme case and far greater than would ever be used on any map where close accuracy is a necessity, and yet even in this case the distortion, although it does exist, and is an absolute necessity, is nevertheless reduced to a minimum by this method.

Numerical Example. Compute by derivation from Table II, for the scale of 1 ÷ 1,000,000 the values for each intersection point of parallels at intervals of 0° 15′ and meridians at every 0° 15′ for 3° east and west of the prime meridian and between the latitudes of 36° and 37°. Draw lightly in pencil the horizontal lines and the perpendiculars, indicating in pencil the computed length of the divisions of the horizontal lines and the lengths of each perpendicular, then connect the corresponding intersections with lines which will represent the parallels and meridians. Draw to a scale of 1 : 100,000.

22. Three Point Problem. Economy in field work is frequently served by employing what is known as the "three point problem" to accomplish certain results. For example, the location of points at which soundings are taken in any body of water require the employment of several extra men with instruments if it is done by the method which is perhaps simplest to plot, namely, to have men and instruments on certain located points on shore where observations are taken at the time of making each sounding. This may be obviated by using two sextants or even a double sextant, which is used by a man on the sounding boat. Observations by the boat observer are taken on three shore signals, and the two angles between each outer signal and the middle signal are measured. With an instrument known as a double sextant, one skillful observer can simultaneously set his sextant to take both readings. This method keeps all members of the party together. No signaling between those in the boat and instrument men on shore is necessary and the economy of time and men is considerable, although it requires special instruments to take the observations and a special instrument later on to plot the work. Unfortunately even this solution has its limitations. If the circle which passes through the three fixed points also passes through the point to be located, then the solution is indeterminable, and if the point to be determined lies *near* the circumference of that circle, the solution, although theoretically determinate, is subject to great inaccuracy. Nevertheless, with suitable care this limiting condition may be avoided and the accuracy under usual conditions is as great as is necessary. The same fundamental principle is employed in plane table surveying when it is desired to set up a plane table at some station which has not been previously plotted on the map, and to determine directly its location by an observation on three stations which have already been plotted. The fundamental principle of the work is illustrated in Figure 15 and depends on the geometrical proposition that, except for a point located on the circumference *a b c*, there is but one position, *x*, from which the lines drawn forming the angles *a x b* and *b x c* will equal any two given values. An inspection of the figure will show that if *x′* were located in the circumference, then the angles *a x′ b* and *b x′ c* would be the same for any location of *x′* in that circumference. If *x* represents, for example, the location of a boat from which the angles *a x b* and *b x c* are measured between flags located

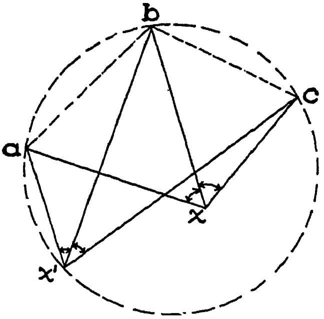


Fig. 15. Three-Points Problem.

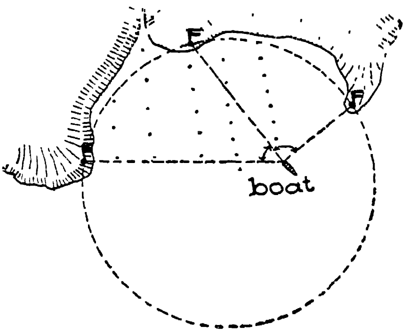


Fig 16. Locating Soundings—Three-Points Problem.

it is only necessary, in order to plot the position of *x*, to have three lines which make the given angles with each other and find by trial such a position for *x* that three such lines will simultaneously pass through the three points. These three lines may be the three arms of a "three-armed protractor" (see Fig. 21) or they may be three lines which make the given angles and which are drawn on a piece of tracing paper, which is used as described below. Of course, very skillful work is required to avoid inaccuracy, but in skilled hands the method is economical and sufficiently accurate for practical use.

In plane table work the problem may be solved similarly. The plane table may be set up over the station whose location is required. A piece of tracing paper may be placed over the regular drawing and three lines may be drawn to the three known and plotted stations. These lines will evidently make the proper angles with each other. It is then only necessary to move this tracing paper to some position, such that the lines will simultaneously pass through the three points and then prick through the tracing paper the location of the desired point.

Another method of solving the same problem with the plane table is that known as Bessel's solution, which is illustrated in Fig. 17. The explanation of this method is far more difficult than the mere work of doing it, which may be readily learned. The reasoning may be best followed by considering a case which has already been worked out—see Fig. 17. *a*, *b* and *c* are points already plotted on the map. *d* is a point whose real location is at first unknown. *e* is a "construction" point, which is also unknown. The method of procedure will be first stated as follows:

The instrument is set up over the point *D* in the field. *A*, *B*, *C* and *D* refer to the actual station positions. (1) The alidade is then set on the line *c a* and the whole table is then revolved until the instrument sights at *A*, when it is clamped. (2) The alidade is then placed with its edge on *c* and so that it sights at the point *B*. Drawing a line, we will have the line *c e*. (3) The alidade is set on the line *a c* and the table is then loosened and turned until the telescope points at *C*. (4) Move the alidade so that the edge passes through *a* and again points at *B* and draw a line. This line *a e* drawn from *a* intersects the line drawn from *c* in the point *e*. (5) Connect *b* and *e*. The required point *d* is somewhere on the line *b e*—or on *b e* produced. By placing the alidade on the line *b e* and turning the whole table until it points at station *B*, the table is properly oriented. The student should note that the location of point *d* is still unknown, but when the table is properly oriented, it is only necessary to place the alidade with its edge on the point *a* and so that it sights at station *A*. A line drawn along the alidade will intersect the line *b e* at the required point *d*. As a check we may turn the alidade so that its edge is on the point *c* and so that the telescope points at station *C*. The edge of the alidade should again pass through the point *d*; the precision with which it does so forms a valuable check on the accuracy of the work done.

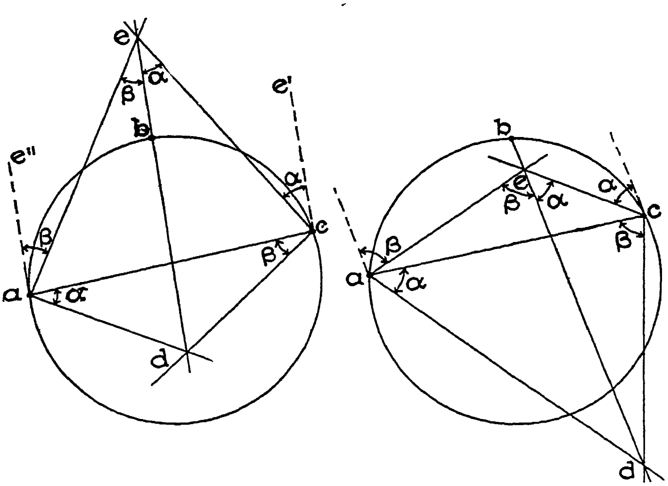


Fig. 17 Bessel's Solution of Three-Point Problem

Explanation. First step: alidade on *c a*, sighting at *A*. Since the instrument is at *D*, the proper direction of the alidade when sighting toward *A* should be *d a* and would be so if the table were properly oriented. Therefore the orientation is in error by the angle α to the *right*. Second step: the line *c e*, which should have the direction *c e′*, makes the angle α with *c e′*. In other words, the line *d b e* makes the angle α with *e c*. Third step: alidade on *a c*, sighting at *C*. Since the instrument is at *D*, the proper direction of the alidade when sighting toward *C* should be *d c*, and would be so if the table were properly oriented. Therefore the orientation is in error by the angle β to the *left*. Fourth step: the line $a\ e$ which should have the direction $a\ e''$, makes the angle β with $a\ e''$. In other words the line $d\ b\ e$ makes the angle β with $e\ a$. Fifth step: d lies somewhere on $b\ e$ or on $b\ e$ produced. Since the table can be accurately oriented by sighting on B with alidade on $d\ b\ e$, the location of d then becomes easy as a problem in "resection" by turning the alidade to sight at either A or C. The alidade will intersect $d\ b$ at d and thus determine the point. The student should note that as d approaches the circumference, e approaches b, the line $b\ e$ becomes very short and since the solution depends on drawing $b\ e$ accurately in direction, it becomes more and more inaccurate as d approaches the circumference. When d is on the circumference, e will coincide with b and the solution is indeterminate. It should also be noted that d and e are interchangeable—*i.e.*, if e were the unknown point, then d, determined by the intersection of $a\ d$ and $c\ d$, would be the point which, when joined with b, would give the line on which the true point e would be located. e would then be determined by resection from a or c.

This method has the theoretical inaccuracy of assuming that lines drawn from various points of the map toward a distant station are parallel. When the scale is very large the error *might* be appreciable with the most careful work, but ordinarily it is impossible to do plane table work with sufficient exactness to detect an error due to this cause. The advantage of this method over the tracing paper method is that although it requires the drawing on the paper of the lines $a\ e$, $b\ e$, and $c\ e$, which are otherwise useless, it does not necessitate the use of tracing paper which might prove inaccurate on account of slipping. When it is found that the unknown point d is nearly, if not exactly, on the circumference of a circle passing through a, b and c, then some other station should be substituted for a, b or c, and ordinarily this will permit the problem to be solved with sufficient accuracy.

23. Topographical Signs. There is a very great advantage in employing colors to represent the topographical signs, but if a map is being prepared for reproduction it is impracticable to use colors unless it is designed to make several impressions, one in each of the various colors, which makes a very expensive map. The systems of topographical signs must therefore be based very largely on whether colors are to be used or whether the whole map must be in black. When one individual map is being made, it is almost as easy to employ colored inks, and this should certainly be done if the map is designed to represent considerable variety of detail. Contour lines are usually drawn with "burnt sienna," a yellowish-brown color which is not so very unlike that of clay and which is quite suggestive. In spite of the added expense, the United States Geological Survey maps have all contours printed in a similar color. Similarly water lines are usually represented with blue lines if color is employed. The United States Geological Survey maps represent the shores of all lakes, rivers, etc., with blue lines. Even the smallest streams which are represented on the map are drawn in blue even though it is but a single line, and swampy districts are represented by a suitable topographical sign, which is also drawn in blue. This practice permits an instant recognition of the character of the sign and prevents any possible confusion of mistaking a stream line for a contour line. Artificial features of the topography are represented in black. By artificial features are meant anything in the nature of constructions such as buildings, railroads, dams, property lines or political boundaries, etc. As stated previously under "scale of map," judgment should be shown whether to endeavor to represent the actual form of buildings to their proper scale. On the United States Geological Survey maps individual dwelling houses are usually represented by a minute square of black even when the scale of the map is as small as one mile to the inch (approximately). Since the maps are drawn very finely and the squares are perhaps not more than $\frac{1}{100}$ of an inch square, they do not necessarily indicate much, if any, exaggeration. Of course, at such scale no attempt is made to give the exact form of the building. The well known representation of a railroad by a single heavy line crossed at frequent intervals with cross lines as if in imitation of cross ties is always recognizable. When the scale is sufficiently large, say 100 feet to the inch, even the two rails may be indicated by a double line rather than a single line. Highways are indicated by double lines which should be spaced at approximately their proper width. Since a neat appearance absolutely requires that these lines should be evenly spaced and exactly parallel, it is preferable to draw these lines with what is called a "road pen," which consists of two ruling pens whose distance apart can be regulated with rapidly drawn in so that both lines will not only be uniform, but are exactly the same distance apart at every point.

Some instruction books have been published with very elaborate systems of topographical signs to represent by distinctive signs every conceivable form of vegetation and also to denote the character of different kinds of factories, mines and other structures. In several cases where a map is made for the distinctive purpose of denoting the location of the various features which pertain to some special industry, these special topographical signs may be justifiable, but their significance should invariably be indicated by a suitable legend inscribed on some part of the map. Although every surveyor should know the significance of the topographical signs which are used to indicate the common features which are generally found on every map, it is useless to expect that the special and little used topographical signs will be familiar to all engineers. Plates I and II show the signs which have been adopted by government use and which therefore may be considered as standard. No others should be placed on a map without some lettering on the map to indicate their character, especially as there is some confusion regarding the method of indicating some of the more uncommon topographical features.

24. Lettering. Many a map which is otherwise good is spoiled by poor lettering. Part III of Mechanical Drawing contains a treatise on lettering, and therefore such instruction will not be repeated here, since this course forms a part of every engineering course. In general it should be said that the style of lettering adopted should be simple and as plain as possible. No fancy lettering should be used except perhaps in the title, and even here some forms of fancy lettering are not only in bad taste but are a useless waste of time and skill. The illustrations of government maps herewith given should be particularly studied with reference to the style of lettering used for different topographical features. Where much lettering is necessary (for example, to indicate the location of numerous towns on a small scale map) the simplest form of line lettering is preferable. When possible all lettering should run horizontal with the base of the map, except the lettering to indicate the name of streams, railroads, etc., which should follow their course. The spacing between letters used to indicate the names of streams and railroads should likewise be strung out to cover a considerable part of the length of

CHARTING DIRECTLY FROM NATURE

the stream or railroad unless that spacing would be so great as to make it difficult to follow the continuity of the letters. The Roman style of lettering is usually adopted on government work, because it is the best in spite of its being the most difficult style to make so that it looks well. The student should cultivate the ability to make lettering in this style. A simple form of line lettering is infinitely preferable to an attempt at fancy lettering, and the draftsman must not consider that he is qualified to do lettering until he can make good letters at least in this simple form.

25. Border Line. A simple border line which consists of a single heavy line of uniform width will add considerably to the neatness of a map, and it should usually be made. When it is specially desired to attract the eye of non-technical people, as in the case of real estate maps, which are made to look as attractive as possible to catch the eye of a possible purchaser, a very elaborate border line may be justifiable, but otherwise it must be considered as a waste of time and skill, and even an evidence of bad taste. It sometimes happens that a farmer will think a great deal more of the map of his farm if it is surrounded by an elaborately executed border line, and since it is policy to sell to a man what he wants and is willing to pay for, it would be justifiable in such cases to consult a book on lettering, which usually contains various specimens of border lines, meridian marks, etc., which will set off a map and make it look very pretty even though it does not add one whit to its real value. The sheets issued by the United States Geological Survey have no border line, and even the large sheets issued by the United States Coast and Geodetic Survey have border lines which are simple and dignified in character.

26. Drawing Paper. Considering that the work on very large maps is very great and sometimes requires a very high grade of labor for weeks and even months, which means that the map represents an expenditure of labor worth several hundred dollars, it is folly to lessen the value of the map by using a cheap grade of paper. A good quality of drawing paper should be quite smooth and yet of such a texture that pencil lines and even light ink lines can be erased from it without necessarily destroying the surface of the paper. For some kinds of work where but very little penciling is required and such may be desirable to use Bristol board, since its surface is very smooth and, with very fine pens, work may be done which is almost equal to engraving, but Bristol board becomes roughened when it is rubbed, and therefore it should not be used when many construction lines are necessary which need to be more or less erased. "Hot pressed" Whatman's paper is the best for map work, since it is smoothest and it can be obtained in large sheets. When it is necessary to make very large maps (four feet square or over) it is advisable to have the paper "mounted" on linen. This becomes almost necessary on account of its greater durability, for a large sheet would be more apt to be torn. Manufacturers of drawing paper make these large sheets by pasting smaller sheets together on the mounting of linen, the joints being beveled and carefully rubbed down so that they are practically invisible and do not give the roughness that would ordinarily be found with an ordinary lap joint. Since such work is done so much better by the manufacturers, it is usually preferable to leave such work in their hands. When it is absolutely essential for one to do his own paper mounting, it may be done as follows: Stretch the cloth on a drawing board, if a sufficiently large one can be found, or even on a smooth floor, and tack it down on all sides, stretching it as much as possible, so that it is perfectly smooth. Trim the edges of the sheets of paper (say four) which are to be joined together, and with a rubber grind down the edges which are to form the joint until they get literally to a feather edge. Cover each sheet thoroughly with paste and then spread it on the cloth as smoothly as is possible in its puckered condition, taking special care that the edges of the sheets overlap properly and that a perfectly smooth joint is made. Sometimes a warm flat iron may be successfully used while the paper is drying to smooth out any wrinkles that have formed. By watching the paper during the process of drying and carefully smoothing out wrinkles until all have disappeared, and then leaving the paper for at least twenty-four hours until it is perfectly dry, it will usually be found that the sheet will be perfectly smooth when it has dried. Only the best quality of paper should be used for such a purpose.

DRAWING INSTRUMENTS.

The chief drawing instruments required for topographical work are very few and simple. A protractor, a scale and a straight edge, with pens and pencils, almost comprise the list—except a few instruments which are used for special purposes.

27. Protractor. The choice of a protractor has already been discussed and it was pointed out that the very small metal protractor frequently included with a set of drawing instruments is almost useless for any practical purpose. It is so small that it is impossible to use it with any degree of accuracy. Celluloid or "xylonite" protractors with a diameter of 6 inches are very useful for plotting topography when the scale of the map is such that the length of the lines plotted is not more than 5 or 6 inches, or say twice the radius. Paper protractors, 14 inches in diameter, printed on Bristol board, which can be bought for 40 cents, are much more useful, and with reasonable care will last long enough to fully pay for their cost. Colby's protractor has the advantage that it can be permanently fixed on the paper by means of weights resting on the ears that project from the circle and a very large number of points may be plotted from one station without the wearisome endeavor to hold an ordinary protractor continuously in a fixed place. Additional scales can be attached to the central rotating plane so that any point can be immediately plotted at its proper distance from the center unless it hap-

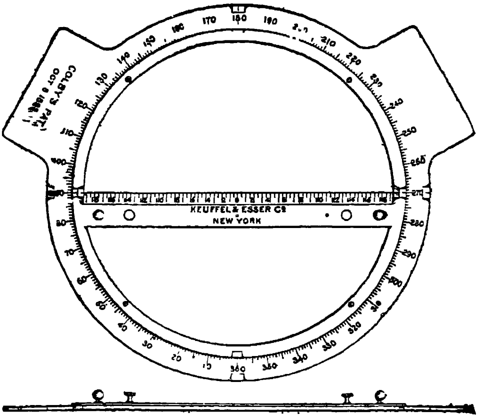


Fig. 18. Colby's Protractor

The instruments cost about $60 with $3.50 extra for extra scales. Metal protractors of the Crozet type have the very great advantage that they can be used in connection with a T-square and do not need to be centered over the station point. After setting the protractor at the proper angle, it is only necessary to slip it along the T-square or along the fixed straight edge, which has been properly clamped on the board, and when any point of the fiducial edge of the protractor reaches the given station, a line may be drawn through the station at the desired angle. Some of these protractors are made with vernier and slow motion screws, so that they will read single minutes of arc, but it requires a degree of accuracy in the drafting work, which is seldom, if ever, attained to draw the lines as accurately as this implies. Such a protractor costs about $40. The Darling, Brown and Sharpe protractor works on the same fundamental principles as the Crozet protractor; it has a vernier reading to 5 minutes of arc and costs only $6.50. It is practically as useful and as accurate as the more expensive forms.

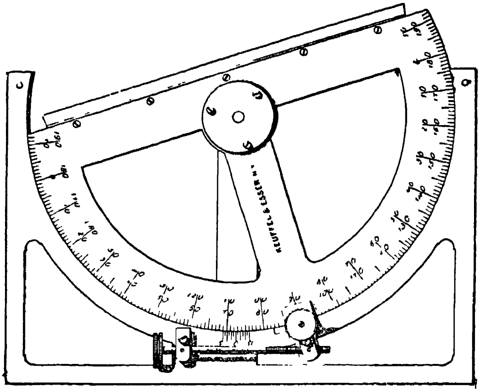


Fig 19. The Crozet Protractor.

28. Scales. Usually "decimal" scales are used for plotting topographical work, since the maps are frequently drawn on a scale of 100 feet, 200 feet, 400 feet, or 500 feet per inch. Therefore a "triangular" decimal scale with its six scales graduated to 10, 20, 30, 40, 50 and 60 parts per inch is in general the most useful for that purpose.

Even if dimensions are scaled off in hundredths of an inch, the simplest plan is to use the "50" scale in which each division means $\frac{1}{50}$ = .02 inch. When maps are drawn to some special scale, as for example, 1: 125000, or 1: 62500, it will facilitate the work (and it will be justifiable) to construct special scales which will permit the distance in feet as measured in the field to be scaled off directly on the map. There is a theoretical argument in favor of using paper scales, the argument being that the paper of the map and the paper scales will vary equally with changes in the hygrometric conditions of the atmosphere. But it must be considered that these changes are very small and almost insignificant unless dampness of the atmosphere is so great that the paper of the map becomes positively puckered, and under such conditions it is perhaps inadvisable to attempt to do any work. Also unless the paper scale was made of the same quality of paper as the map, which is not very likely, the coefficient would not be the same. It is thus seen that the advantages of using paper scales are more fancied than real, while their disadvantages are many. Where the work to be done is very extensive and it must all be done at the same scale, there is a considerable advantage in using a "flat" scale rather than a "triangular" scale, since under such conditions the five other

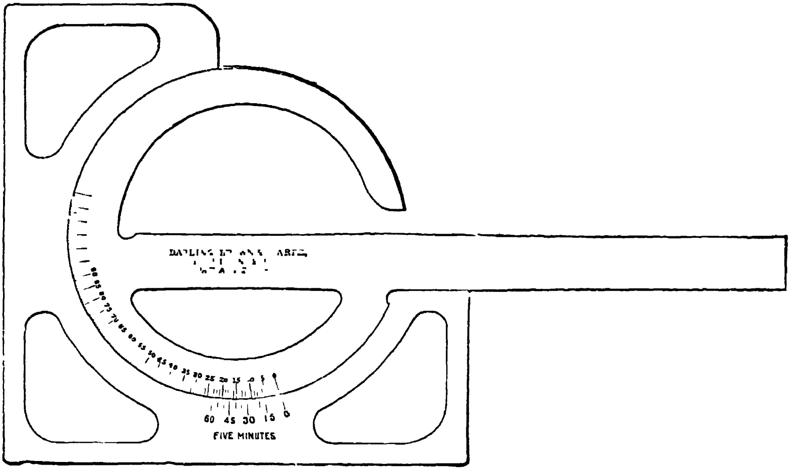


Fig. 20. Steel Protractor.

scales of the triangular scale are useless and a blunder is frequently caused by inadvertently using the wrong scale. Metal scales have the advantage of extreme durability but the disadvantage of tending to soil the paper. The boxwood scales having a natural wood color are the most common, but they grow darker with age and with handling and the graduations become so dim that it is difficult to use them. Ivory scales are very liable to warp and shrink, and although they have the advantage of distinctness and a neat appearance, their tendency to warp is a decided disadvantage. Probably the best scales are those which are made of boxwood but which have the beveled edges covered with a material which resembles ivory, which permanently remains white, and which does not shrink as ivory does.

29. **Straight-Edge.** It is not a good plan to use the edge of the scale as a straight-edge. The rubbing of the pencil over the edge of the scale will soil it. It is always better to have two triangles, so that lines at right angles may be readily drawn in as is necessary in showing the sides of buildings. A very long straight-edge is occasionally necessary. When a very long line is to be drawn perfectly straight and no straight-edge of that length is at hand, the line may be drawn by stretching a silk thread between the two terminal points and then marking, with a pencil, points at such frequent intervals that the small straight-edge available may span those distances. It is needless to say that extreme care is absolutely essential in drawing a line by this method. Even well-made straight-edges may become warped and lose their straightness. To test a straight-edge, carefully draw a line with it and reverse the straight-edge end to end and note how closely it coincides with the line drawn If no error is observable, the edge is probably straight, but it is conceivable that a straight-edge might not be straight and yet stand this test, for the edge might have a form somewhat similar to an *S*, that is, it might be curved symmetrically about its center so that when reversed the edge would again coincide with the line drawn, but this is a very improbable condition.

30. **Pens and Pencils.** The lead used in the pencils should be of good quality and should be neither too hard nor too soft. A very soft pencil will have its point worn off very rapidly and the lead will smudge the paper. If the lead is too hard, it requires so much pressure on the paper to produce a visible mark that the paper is

actually indented, and if it becomes necessary to erase the line, the paper is indelibly impressed with a groove where the line was drawn. It is therefore necessary that all lines and marks should be drawn with a very light pressure of the pencil, which practically requires that the lead should be soft enough so that even a light pressure will make a mark that is easily visible. Although some of the inking work is done with a ruling pen, the great bulk of topographical mapping is done with a nib pen. The pen should be sharply pointed, so that it is capable of making a very fine line. Although much of such work is done with "crow quill" pens, a larger pen (say Gillott's No 303) will serve equally well and in some respects is more useful. The rubber used should be a soft rubber rather than hard, since hard rubber will scratch the paper. A "sponge rubber" to clean the drawing when it is finished is almost a necessity.

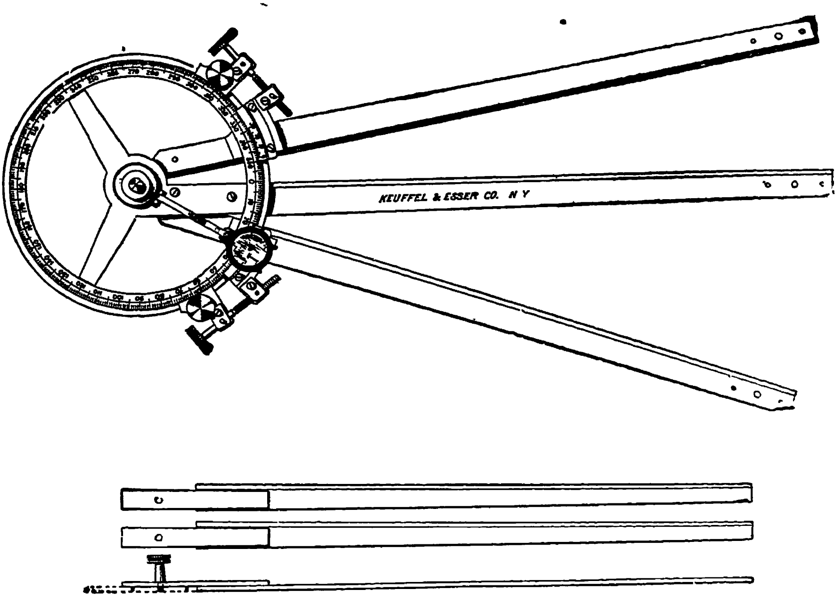


Fig. 21. Three-Armed Protractor.

31. Three-Armed Protractor. Among the special drawing instruments which are occasionally needed is a "three-armed protractor." This instrument is used when plotting points which have been observed on the principle of the "three-point problem." The instrument consists essentially of a protractor having one fixed arm and two movable arms which may be set at the two given angles with the middle arm. Then when the fiducial edges of these three arms are simultaneously placed on the three given points, the center of the instrument *must* be located at the point which represents on the map the point from which observations were taken, and this point is pricked through the center of the instrument. As previously explained, the operation of this instrument depends on the fundamental principle that (unless the point to be located happens to lie in the circumference of the circle which passes through the three given points sighted at) there is only one position from which it is possible to draw three lines which make the given angles with each other through those three points. The angles are therefore laid off by moving the two outer arms so that they make the required angle with the middle fixed arm; then with the arms clamped in position, the whole instrument is shifted around on the paper until the fiducial edges of the rules simultaneously pass through the three fixed points. It is needless to say that great care and accuracy is necessary, and especially so if the point to be determined lies near the circumference of the circle passing through the three fixed points, for it will be found that a very slight inaccuracy

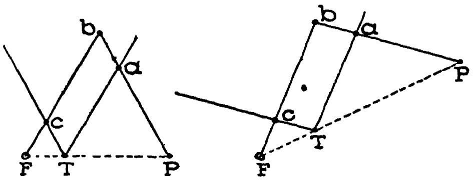


Fig. 22. Principle of Pantograph.

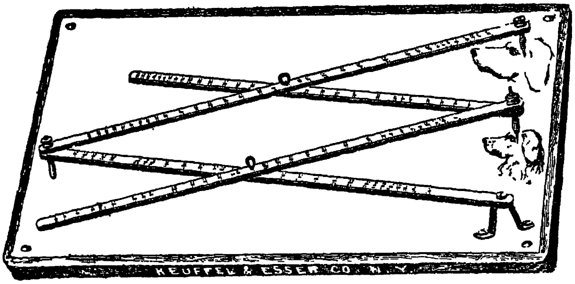

KEUFFEL & ESSER CO. N.Y.

Fig 23 Wooden Pantograph.

will cause a considerable variation in the location of the unknown point; or in other words, there will be a considerable area from any point of which the three lines will pass approximately through the three fixed points.

32. Pantograph. The pantograph is a very useful and essential instrument when drawings are to be re-drawn on a different scale. It frequently happens that field drawings, especially when made with plane table, are drawn to a larger scale than is used for the final engraved maps. The principle of the pantograph is as shown in Fig. 22, where *F* represents the fixed point, *T* a tracing point and *P* a pencil or marker. *F*, *T* and *P* must *always* be in a straight line. *F c T* and *F b P* must always be *similar* triangles. In this instrument they are *isosceles* triangles. *a b c T* is always a parallelogram. The ratio *P F* : *T F* is the ratio of enlargement. To reduce, transpose *P* and *T*; *i.e.*, the marker will be placed at *T* and the tracer at *P*. The diagram shows the fundamental principles, but in practice accuracy requires very fine workmanship and an expensive instrument. Very simple instruments are sometimes made by combining four sticks as shown in the sketch, using rivets at the joints, but the friction of the joints makes such simple instruments inaccurate. Fig. 23 shows an instrument in this simple form, which may be bought for $1.75. Fig 24 shows another instrument called a precision pantograph, which has the same essential geometrical principle, although the mechanical construction has a somewhat different outline and the material, which is entirely of metal, has the very finest quality of workmanship. The

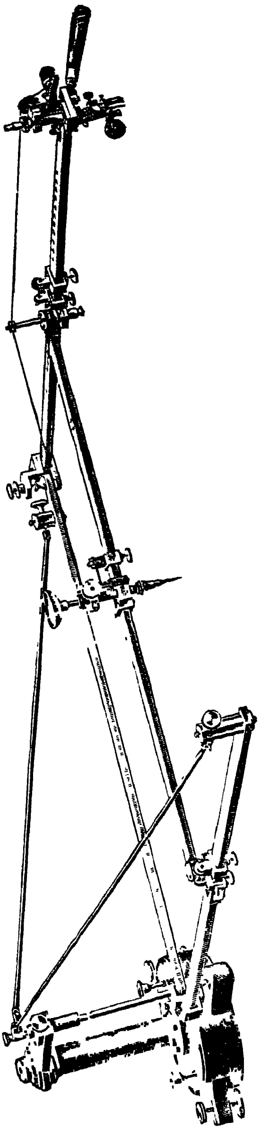


Fig. 24. Metal Pantograph.

33. Stadia Slide Rules. Figs. 25, 26, 27 show three forms of stadia slide rules, the use of which is described in Sections 15 and 16. Any surveyor who has any great amount of stadia surveying to do will be wise to utilize the stadia slide rule to facilitate the stadia reduction, since the cost of the instrument is saved in a very few days and the money necessarily wasted by a neglect to employ this device is very false economy.

34. Polar Planimeter. This instrument is used to measure directly from any drawing the area of a given figure no matter how irregular it may be. Although they are somewhat expensive, the one illustrated in Fig. 28 costing $28, they are capable of marvellous exactness, and, considering the simplicity of their mechanism, it is a wonderful mathematical invention that such a simple mechanism would permit the recording on an index of the exact area of an irregular figure simply by tracing the pointer around the perimeter of the figure no matter how irregular it may be. The polar planimeter, as its name implies, is worked around some fixed point, called the "pole", which in practice means a needle point having a shoulder which is pricked into the paper exactly as is done with the needle point of a compass. The planimeter consists of two arms, the *pole arm*, which carries the fixed point and which is hinged to the *tracer arm*, which carries the tracing point and the measuring wheel with

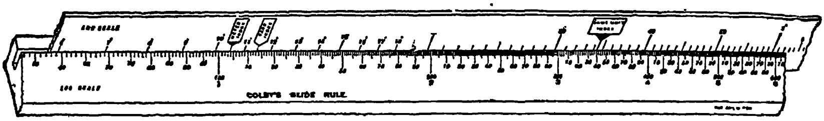


Fig. 25. Colby's Stadia Slide Rule.

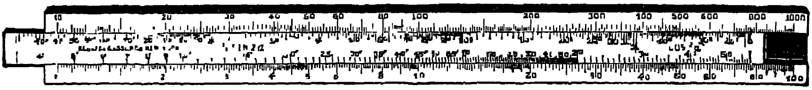


Fig. 26 K & E Stadia Slide Rule

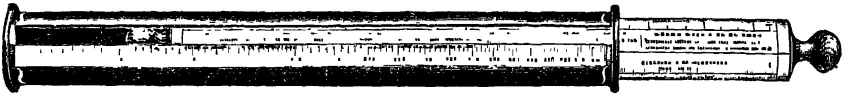


Fig. 27 Webb's Stadia Slide Rule.

the mechanism for indicating the area. The hinge will permit the tracing point to be moved away from the pole by an amount nearly equal to the combined length of the two arms, and if it is possible to fix the pole at some point from which every point in the perimeter of the figure may be reached by the tracer, then the area of the figure may be computed by a single setting of the instrument. If the figure is too large for this, then the area may be divided by lines, which are preferably straight, although not necessarily so, into two or more partial areas, and the area of each section may be separately computed. Perhaps the most common use of the polar planimeter is to compute the area of indicator cards. This has already been described in the course of instruction in steam engineering under the title "Steam Engine Indicators", but the instrument also has another important use in the measurement of irregular ground areas, especially those which have been plotted by stadia methods rather than by the courses and distances of the boundary lines and which therefore cannot be computed by the common rule of a summation of products of latitudes and departures. It is thus possible to compute an area by surveying it rapidly by the stadia method, plotting it with desired accuracy at a suitable scale and then measuring the area with a polar planimeter.

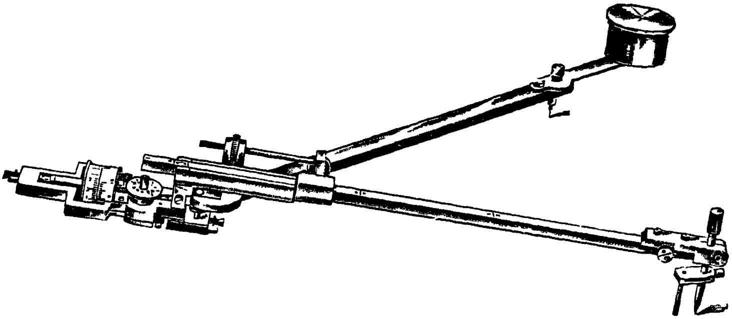


Fig 28 Polar Planimeter.

THEORY OF THE POLAR PLANIMETER.

1. **Zero Circle.** When the planimeter is in the position $P H W C$, Fig. 29, the plane of the wheel, which is perpendicular to the axis $P H$, passes through C. If the instrument is revolved about C, with the angle $W H C$ ($= a_0$) always *constant*, the motion of the wheel over the paper will have no component in the direction of its

plane and the wheel will not revolve. The pointer in this position describes the "zero circle".

2. **Combined Sliding and Rolling.** When the planimeter is in the position P' H' W' C and is revolved about C, with the angle W' H' C ($= \alpha_1$) always *constant*, the wheel will have a combined sliding and rolling motion. For an infinitesimal movement W' b the wheel will roll an amount W' a and slide perpendicular to its plane an amount a b. When rolling in this direction, the movement is called *negative*.

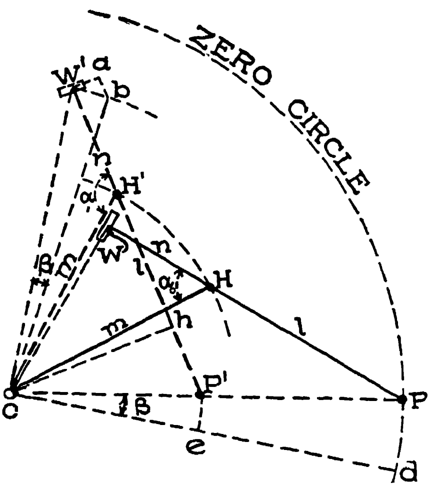


Fig. 29. Polar Planimeter Diagram.

3. **Radial Motion.** When the point P is moved from P to P', the wheel W will both slide and roll, but its rolling will all be in a *negative* direction.

4. **Reversed Radial Motion.** If the pointer were to move back from P' to P the wheel would again slide and roll in precisely the same amounts but in contrary directions, and when it reached P it would have identically the same position and the reading of the index would be identical with the previous reading at P.

5. If the pointer were to move from e to d, the amount and direction of both the slipping and rolling would be the same as when it moved from P' to P.

6. If the pointer were to start from P, move to P', thence to e, thence to d, and thence back to P, the resultant rolling of the wheel is the same as that for the line P' e alone; for the rolling for d P is zero (¶ 1) and the rolling for P P' will be just neutralized by that for e d. (¶'s 3, 4 and 5.)

7. Therefore when the pointer is moved to the right on the arc of a circle within the zero circle about C as a center the indication is *negative* and is the same as if the pointer moved around the area

included between the arc, the corresponding arc of the zero circle, and the including radii.

8. If the pointer moved in the opposite direction the indication would be the same in amount but *positive.*

9. By similar demonstrations, similar facts may be shown for any other elementary area except that

(*a*) when the pointer is *outside* the zero circle and moving to the *right,* the indication is *positive,* and

(*b*) when *outside* and moving to the *left,* the indication is *negative.*

10. The perimeter of any area may be considered as made up of a combination of *infinitesimal* arcs and radial lines having the fixed point of the planimeter as center. Its total area is the *algebraic* sum of all the infinitesimal areas lying between *each* arc and the zero circle.

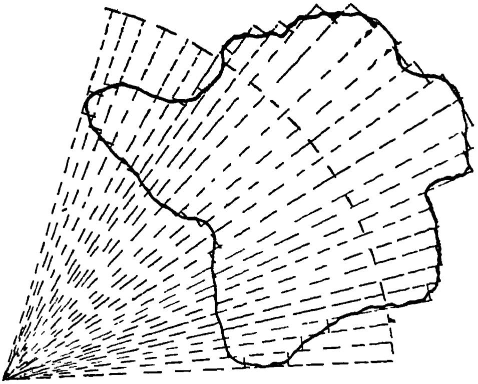


Fig. 30 Irregular Figure Divided Into Elementary Radial Areas.

11. If the pointer of the planimeter moves around each infinitesimal area in turn in such manner that when moving on the perimeter it moves in the same direction as though moving continuously around the perimeter only, the pointer will move over all interior lines an *even* number of times in *opposite* directions. Therefore the *accumulated* registration of the wheel will be the same as though it moved on the perimeter only, for all registrations on interior lines will be nuetralized by the equal motion on them in opposite directions. (¶'s 4 and 6.)

12. Referring to Fig. 29, $P'c = CP' \times \beta =$

$$\sqrt{m^2 + l^2 - 2\,ml \cos a_1} \times \beta.$$

$W'b = CW' \times \beta$. The rolling of the wheel $= W'a$ (¶ 2).

$$W'a = \frac{W'h \times W'b}{CW'} = (n - m \cos a_1)\,\beta, \text{ since } \frac{W'b}{CW'} = \beta$$

and $W'h = (n - m \cos a_1)$

$$\text{Area } P\,P'\,e\,d = \frac{1}{2}(\overline{PC} \times \beta)\,\overline{P'C} - \frac{1}{2}(\overline{P'C} \times \beta)\,\overline{P'C}$$

$$= \frac{1}{2}\beta\,(\overline{PC}^2 - \overline{P'C}^2)$$

$$= \frac{1}{2}\beta\left((l^2 + n^2 + 2nl) + (m^2 - n^2) - (m^2 + l^2 + 2ml\cos a_1)\right)$$

$$= \beta l\,(n - m\cos a_1)$$

$$= l \times W'a \text{ (i.e., } l \text{ times the rolling of the wheel.)}$$

13. When the pointer moves around an elementary area bounded by an arc, by the corresponding arc of the zero circle and by the two bounding radial lines (all having "C" as center) the resultant motion of the wheel is the same as though it moved on the arc alone (¶ 6); the wheel rolls a distance equal to the area divided by "l" (¶ 12). If it moved in turn around each elementary area of a large area, the resultant motion of the wheel would be the same as though it moved continuously around the perimeter of the larger area (¶'s 10, 11) and therefore the total resultant motion of the wheel will equal the area of the figure divided by "l".

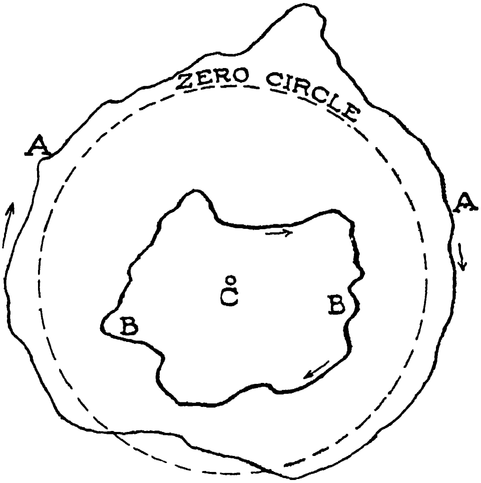


Fig. 31 Fixed Center Inside Area.

14. Therefore if $c =$ the circumference of the wheel, n the number of turns recorded by the index, and l the length of the arm from H to P, then Area $= l\,n\,c$.

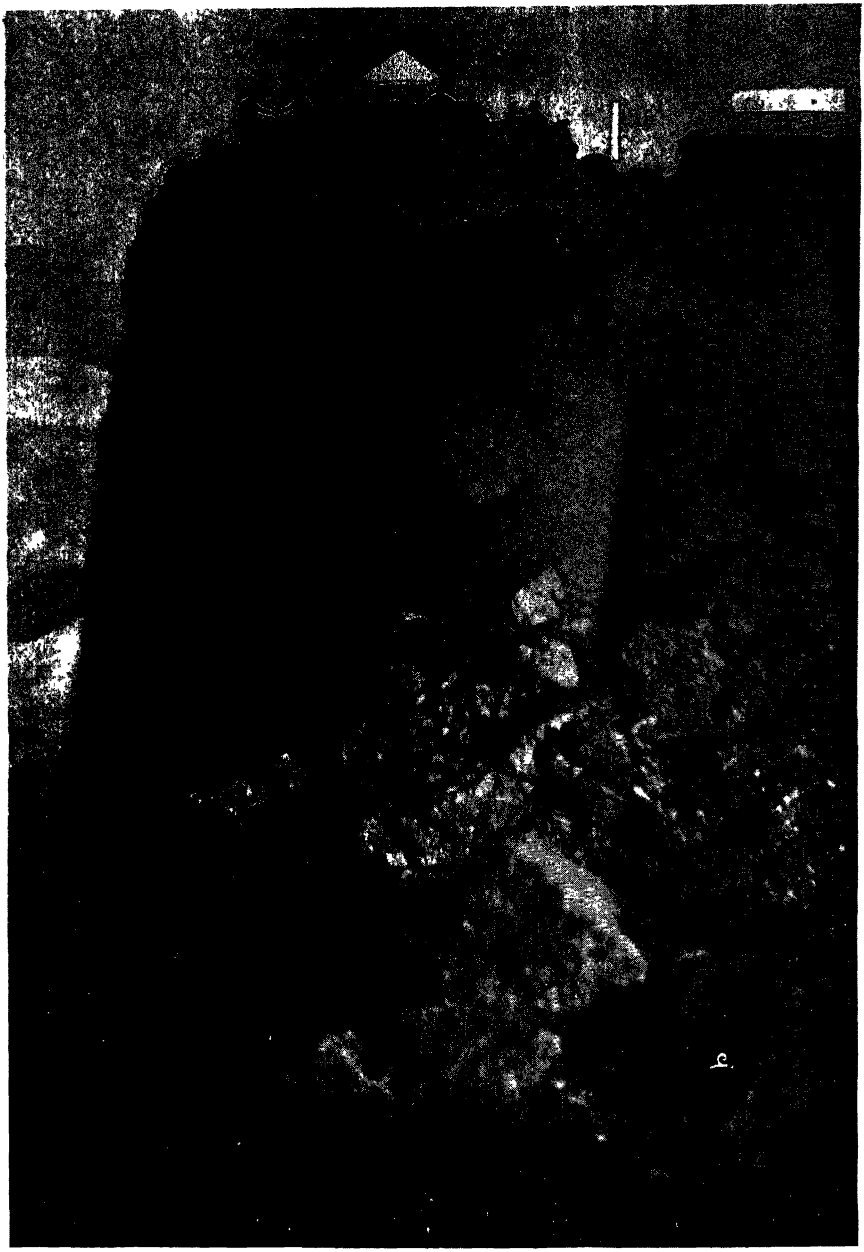
15. Fixed Center Inside the Figure. If the pointer is moved around the perimeter A, Fig. 31, to the *right*, the indication will be *positive* (¶ 9) but will indicate only the area between A and the zero circle. Therefore the total area will equal the indicated area ($l\,n\,c$) plus the area of the zero circle $\left(\pi\,(m^2 + l^2 + 2\,n\,l)\right)$. If the pointer is moved to the *right* around perimeter B, the record will be *negative* (¶ 9) and will correspond to the area between B and the zero circle. Therefore the *algebraic* sum (the numerical difference) of the record reading and the area of the zero circle will give the true area of B.

16. General Rule. *Always* move the pointer to the *right.* If "C", the pole, is within the figure, add the area of the zero circle (algebraically) to the indicated result. If the pole is outside the figure the area is given by the product $l\,n\,c$. The scale on the recording wheel is so graduated as to give the result directly. The area of the zero circle is usually indicated on the instrument, but it may be determined experimentally by finding the distance from the pointer to the fixed center, which will cause no rotation of the index wheel when the pointer is swung about the center. This is the radius of the zero circle. Its area should be reduced to the scale of the drawing.

TABLE I.

Horizontal Distances and Elevations from Stadia Readings.

Minutes	0°		1°		2°		3°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	100.00	0.00	99.97	1.74	99.88	3.49	99.73	5.23
2	100.00	0.06	99.97	1.80	99.87	3.55	99.72	5.28
4	100.00	0.12	99.97	1.86	99.87	3.60	99.71	5.34
6	100.00	0.17	99.96	1.92	99.87	3.66	99.71	5.40
8	100.00	0.23	99.96	1.98	99.86	3.72	99.70	5.46
10	100.00	0.29	99.96	2.04	99.86	3.78	99.69	5.52
12	100.00	0.35	99.96	2.09	99.85	3.84	99.69	5.57
14	100.00	0.41	99.95	2.15	99.85	3.90	99.68	5.63
16	100.00	0.47	99.95	2.21	99.84	3.95	99.68	5.69
18	100.00	0.52	99.95	2.27	99.84	4.01	99.67	5.75
20	100.00	0.58	99.95	2.33	99.83	4.07	99.66	5.80
22	100.00	0.64	99.94	2.38	99.83	4.13	99.66	5.86
24	100.00	0.70	99.94	2.44	99.82	4.18	99.65	5.92
26	99.99	0.76	99.94	2.50	99.82	4.24	99.64	5.98
28	99.99	0.81	99.93	2.56	99.81	4.30	99.63	6.04
30	99.99	0.87	99.93	2.62	99.81	4.36	99.63	6.09
32	99.99	0.93	99.93	2.67	99.80	4.42	99.62	6.15
34	99.99	0.99	99.93	2.73	99.80	4.48	99.62	6.21
36	99.99	1.05	99.92	2.79	99.79	4.53	99.61	6.27
38	99.99	1.11	99.92	2.85	99.79	4.59	99.60	6.33
40	99.99	1.16	99.92	2.91	99.78	4.65	99.59	6.38
42	99.99	1.22	99.91	2.97	99.78	4.71	99.59	6.44
44	99.98	1.28	99.91	3.02	99.77	4.76	99.58	6.50
46	99.98	1.34	99.90	3.08	99.77	4.82	99.57	6.56
48	99.98	1.40	99.90	3.14	99.76	4.88	99.56	6.61
50	99.98	1.45	99.90	3.20	99.76	4.94	99.56	6.67
52	99.98	1.51	99.89	3.26	99.75	4.99	99.55	6.73
54	99.98	1.57	99.89	3.31	99.74	5.05	99.54	6.78
56	99.97	1.63	99.89	3.37	99.74	5.11	99.53	6.84
58	99.97	1.69	99.88	3.43	99.73	5.17	99.52	6.90
60	99.97	1.74	99.88	3.49	99.73	5.23	99.51	6.96
$(f+c)=0.75$	0.75	0.01	0.75	0.02	0.75	0.03	0.75	0.05
$(f+c)=1.00$	1.00	0.01	1.00	0.03	1.00	0.04	1.00	0.06



TRIANGULATION STATION ON MOUNT UNCOMPAHGRE. ELEVATION 14,300 FEET

Courtesy of U S Coast and Geodetic Survey, Washington, D C.

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	4°		5°		6°		7°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	99.51	6.96	99.24	8.68	98.91	10.40	98.51	12.10
2	99.51	7.02	99.23	8.74	98.90	10.45	98.50	12.15
4	99.50	7.07	99.22	8.80	98.88	10.51	98.48	12.21
6	99.49	7.13	99.21	8.85	98.87	10.57	98.47	12.26
8	99.48	7.19	88.20	8.91	98.86	10.62	98.46	12.32
10	99.47	7.25	99.19	8.97	98.85	10.68	98.44	12.38
12	99.46	7.30	99.18	9.03	98.83	10.74	98.43	12.43
14	99.46	7.36	99.17	9.08	98.82	10.79	98.41	12.49
16	99.45	7.42	99.16	9.14	98.81	10.85	98.40	12.55
18	99.44	7.48	99.15	9.20	98.80	10.91	98.39	12.60
20	99.43	7.53	99.14	9.25	98.78	10.96	98.37	12.66
22	99.42	7.59	99.13	9.31	98.77	11.02	98.36	12.72
24	99.41	7.65	99.11	9.37	98.76	11.08	98.34	12.77
26	99.40	7.71	99.10	9.43	98.74	11.13	98.33	12.83
28	99.39	7.76	99.09	9.48	98.73	11.19	98.31	12.88
30	99.38	7.82	99.08	9.54	98.72	11.25	98.29	12.94
32	99.38	7.88	99.07	9.60	98.71	11.30	98.28	13.00
34	99.37	7.94	99.06	9.65	98.69	11.36	98.27	13.05
36	99.36	7.99	99.05	9.71	98.68	11.42	98.25	13.11
38	99.35	8.05	99.04	9.77	98.67	11.47	98.24	13.17
40	99.34	8.11	99.03	9.83	98.65	11.53	98.22	12.22
42	99.33	8.17	99.01	9.88	98.64	11.59	98.20	13.28
44	99.32	8.22	99.00	9.94	98.63	11.64	98.19	13.33
46	99.31	8.28	98.99	10.00	98.61	11.70	98.17	13.39
48	99.30	8.34	98.98	10.05	98.60	11.76	98.16	12.45
50	99.29	8.40	98.97	10.11	98.58	11.81	98.14	13.50
52	99.28	8.45	98.96	10.17	98.57	11.87	98.13	13.56
54	99.27	8.51	98.94	10.22	98.56	11.93	98.11	13.61
56	99.26	8.57	98.93	10.28	98.54	11.98	98.10	13.67
58	99.25	8.63	98.92	10.34	98.53	12.04	98.08	13.73
60	99.24	8.68	98.91	10.40	98.51	12.10	98.06	13.78
(f+c)=0.75	0.75	0.06	0.75	0.07	0.75	0.08	0.74	0.10

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	8°		9°		10°		11°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	98.06	13.78	97.55	15.45	96.98	17.10	96.36	18.73
2	98.05	13.84	97.53	15.51	96.96	17.16	96.34	18.78
4	98.03	13.89	97.52	15.56	96.94	17.21	96.32	18.84
6	98.01	13.95	97.50	15.62	96.92	17.26	96.29	18.89
8	98.00	14.01	97.48	15.67	96.90	17.32	96.27	18.95
10	97.98	14.06	97.46	15.73	96.88	17.37	96.25	19.00
12	97.97	14.12	97.44	15.78	96.86	17.43	96.23	19.05
14	97.95	14.17	97.43	15.84	96.84	17.48	96.21	19.11
16	97.93	14.23	97.41	15.89	96.82	17.54	96.18	19.16
18	97.92	14.28	97.39	15.95	96.80	17.59	96.16	19.21
20	97.90	14.34	97.37	16.00	96.78	17.65	96.14	19.27
22	97.88	14.40	97.35	16.06	96.76	17.70	96.12	19.32
24	97.87	14.45	97.33	16.11	96.74	17.76	96.09	19.38
26	97.85	14.51	97.31	16.17	96.72	17.81	96.07	19.43
28	97.83	14.56	97.29	16.22	96.70	17.86	96.05	19.48
30	97.82	14.62	97.28	16.28	96.68	17.92	96.03	19.54
32	97.80	14.67	97.26	16.33	96.66	17.97	96.00	19.59
34	97.78	14.73	97.24	16.39	96.64	18.03	95.98	19.64
36	97.76	14.79	97.22	16.44	96.62	18.08	95.96	19.70
38	97.75	14.84	97.20	16.50	96.60	18.14	95.93	19.75
40	97.73	14.90	97.18	16.55	96.57	18.19	95.91	19.80
42	97.71	14.95	97.16	16.61	96.55	18.24	95.89	19.86
44	97.69	15.01	97.14	16.66	96.53	18.30	95.86	19.91
46	97.68	15.06	97.12	16.72	96.51	18.35	95.84	19.96
48	97.66	15.12	97.10	16.77	96.49	18.41	95.82	20.02
50	97.64	15.17	97.08	16.83	96.47	18.46	95.79	20.07
52	97.62	15.23	97.06	16.88	96.45	18.51	95.77	20.12
54	97.61	15.28	97.04	16.94	96.42	18.57	95.75	20.18
56	97.59	15.34	97.02	16.99	96.40	18.62	95.72	20.23
58	97.57	15.40	97.00	17.05	96.38	18.68	95.70	20.28
60	97.55	15.45	96.98	17.10	96.36	18.73	95.68	20.34
(f+c)=0.75	0.74	0.11	0.74	0.12	0.74	0.14	0.73	0.15
(f+c)=1.00	0.99	0.15	0.99	0.16	0.98	0.18	0.98	0.20

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	12°		13°		14°		15°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev
0	95.68	20.34	94.94	21.92	94.15	23.47	93.30	25.00
2	95.65	20.39	94.91	21.97	94.12	23.52	93.27	25.05
4	95.63	20.44	94.89	22.02	94.09	23.58	93.24	25.10
6	95.61	20.50	94.86	22.08	94.07	23.63	93.21	25.15
8	95.58	20.55	94.84	22.13	94.04	23.68	93.18	25.20
10	95.56	20.60	94.81	22.18	94.01	23.73	93.16	25.25
12	95.53	20.66	94.79	22.23	93.98	23.78	93.13	25.30
14	95.51	20.71	94.76	22.28	93.95	23.83	93.10	25.35
16	95.49	20.76	94.73	22.34	93.93	23.88	93.07	25.40
18	95.46	20.81	94.71	22.39	93.90	23.93	93.04	25.45
20	95.14	20.87	94.68	22.44	93.87	23.99	93.01	25.50
22	95.41	20.92	94.66	22.49	93.84	24.04	92.98	25.55
24	95.39	20.97	94.63	22.54	93.81	24.09	92.95	25.60
26	95.36	21.03	94.60	22.60	93.79	24.14	92.92	25.65
28	95.34	21.08	94.58	22.65	93.76	24.19	92.89	25.70
30	95.32	21.13	94.55	22.70	93.73	24.24	92.86	25.75
32	95.29	21.18	94.52	22.75	93.70	24.29	92.83	25.80
34	95.27	21.24	94.50	22.80	93.67	24.34	92.80	25.85
36	95.24	21.29	94.47	22.85	93.65	24.39	92.77	25.90
38	95.22	21.34	94.44	22.91	93.62	24.44	92.74	25.95
40	95.19	21.39	94.42	22.96	93.59	24.49	92.71	26.00
42	95.17	21.45	94.39	23.01	93.56	24.55	92.68	26.05
44	95.14	21.50	94.36	23.06	93.53	24.60	92.65	26.10
46	95.12	21.55	94.34	23.11	93.50	24.65	92.62	26.15
48	95.09	21.60	94.31	23.16	93.47	24.70	92.59	26.20
50	95.07	21.66	94.28	23.22	93.45	24.75	92.56	26.25
52	95.04	21.71	94.26	23.27	93.42	24.80	92.53	26.30
54	95.02	21.76	94.23	23.32	93.39	24.85	92.49	26.35
56	94.99	21.81	94.20	23.37	93.36	24.90	92.46	26.40
58	94.97	21.87	94.17	23.42	93.33	24.95	92.43	26.45
60	94.94	21.92	94.15	23.47	93.30	25.00	92.40	26.50
(f+c)=0.75	0.73	0.16	0.73	0.17	0.73	0.19	0.72	0.20

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	16°		17°		18°		19°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	92.40	26.50	91.45	27.96	90.45	29.39	89.40	30.78
2	92.37	26.55	91.42	28.01	90.42	29.44	89.36	30.83
4	92.34	26.59	91.39	28.06	90.38	29.48	89.33	30.87
6	92.31	26.64	91.35	28.10	90.35	29.53	89.29	30.92
8	92.28	26.69	91.32	28.15	90.31	29.58	89.26	30.97
10	92.25	26.74	91.29	28.20	90.28	29.62	89.22	31.01
12	92.22	26.79	91.26	28.25	90.24	29.67	89.18	31.06
14	92.19	26.84	91.22	28.30	90.21	29.72	89.15	31.10
16	92.15	26.89	91.19	28.34	90.18	29.76	89.11	31.15
18	92.12	26.94	91.16	28.39	90.14	29.81	89.08	31.19
20	92.09	26.99	91.12	28.44	90.11	29.86	89.04	31.24
22	92.06	27.04	91.09	28.49	90.07	29.90	89.00	31.28
24	92.03	27.09	91.06	28.54	90.04	29.95	88.96	31.33
26	92.00	27.13	91.02	28.58	90.00	30.00	88.93	31.38
28	91.97	27.18	90.99	28.63	89.97	30.04	88.89	31.42
30	91.93	27.23	90.96	28.68	89.93	30.09	88.86	31.47
32	91.90	27.28	90.92	28.73	89.90	30.14	88.82	31.51
34	91.87	27.33	90.89	28.77	89.86	30.19	88.78	31.56
36	91.84	27.38	90.86	28.82	89.83	30.23	88.75	31.60
38	91.81	27.43	90.82	28.87	89.79	30.28	88.71	31.65
40	91.77	27.48	90.79	28.92	89.76	30.32	88.67	31.69
42	91.74	27.52	90.76	28.96	89.72	30.37	88.64	31.74
44	91.71	27.57	90.72	29.01	89.69	30.41	88.60	31.78
46	91.68	27.62	90.69	29.06	89.65	30.46	88.56	31.83
48	91.65	27.67	90.66	29.11	89.61	30.51	88.53	31.87
50	91.61	27.72	90.62	29.15	89.58	30.55	88.49	31.92
52	91.58	27.77	90.59	29.20	89.54	30.60	88.45	31.96
54	91.55	27.81	90.55	29.25	89.51	30.65	88.41	32.01
56	91.52	27.86	90.52	29.30	89.47	30.69	88.38	32.05
58	91.48	27.91	90.48	29.34	89.44	30.74	88.34	32.09
60	91.45	27.96	90.45	29.39	89.40	30.78	88.30	32.14
$(f+c)=0.75$	0.72	0.21	0.72	0.23	0.71	0.24	0.71	0.25
$(f+c)=1.00$	0.86	0.28	0.95	0.30	0.95	0.32	0.94	0.33

TABLE I.
(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	20°		21°		22°		23°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	88.30	32.14	87.16	33.46	85.97	34.73	84.73	35.97
2	88.26	32.18	87.12	33.50	85.93	34.77	84.69	36.01
4	88.23	32.23	87.08	33.54	85.89	34.82	84.65	36.05
6	88.19	32.27	87.04	33.59	85.85	34.86	84.61	36.09
8	88.15	32.32	87.00	33.63	85.80	34.90	84.57	36.13
10	88.11	32.36	86.96	33.67	85.76	34.94	84.52	36.17
12	88.08	32.41	86.92	33.72	85.72	34.98	84.48	36.21
14	88.04	32.45	86.88	33.76	85.68	35.02	84.44	36.25
16	88.00	32.49	86.84	33.80	85.64	35.07	84.40	36.29
18	87.96	32.54	86.80	33.84	85.60	35.11	84.35	36.33
20	87.93	32.58	86.77	33.89	85.56	35.15	84.31	36.37
22	87.89	32.63	86.73	33.93	85.52	35.19	84.27	36.41
24	87.85	32.67	86.69	33.97	85.48	35.23	84.23	36.45
26	87.81	32.72	86.65	34.01	85.44	35.27	84.18	36.49
28	87.77	32.76	86.61	34.06	85.40	35.31	84.14	36.53
30	87.74	32.80	86.57	34.10	85.36	35.36	84.10	36.57
32	87.70	32.85	86.53	34.14	85.31	35.40	84.06	36.61
34	87.66	32.89	86.49	34.18	85.27	35.44	84.01	36.65
36	87.62	32.93	86.45	34.23	85.23	35.48	83.97	36.69
38	87.58	32.98	86.41	34.27	85.19	35.52	83.93	36.73
40	87.54	33.02	86.37	34.31	85.15	35.56	83.89	36.77
42	87.51	33.07	86.33	34.35	85.11	35.60	83.84	36.80
44	87.47	33.11	86.29	34.40	85.07	35.64	83.80	36.84
46	87.43	33.15	86.25	34.44	85.02	35.68	83.76	36.88
48	87.39	33.20	86.21	34.48	84.98	35.72	83.72	36.92
50	87.35	33.24	86.17	34.52	84.94	35.76	83.67	36.96
52	87.31	33.28	86.13	34.57	84.90	35.80	83.63	37.00
54	87.27	33.33	86.09	34.61	84.86	35.85	83.59	37.04
56	87.24	33.37	86.05	34.65	84.82	35.89	83.54	37.08
58	87.20	33.41	86.01	34.69	84.77	35.93	83.50	37.12
60	87.16	33.46	85.97	34.73	84.73	35.97	83.46	37.16
$(f+c)=0.75$	0.70	0.26	0.70	0.27	0.69	0.29	0.69	0.30

TABLE I.
(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	24°		25°		26°		27°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	83.46	37.16	82.14	38.30	80.78	39.40	79.39	40.45
2	83.41	37.20	82.09	38.34	80.74	39.44	79.34	40.49
4	83.37	37.23	82.05	38.38	80.69	39.47	79.30	40.52
6	83.33	37.27	82.01	38.41	80.65	39.51	79.25	40.55
8	83.28	37.31	81.96	38.45	80.60	39.54	79.20	40.59
10	83.24	37.35	81.92	38.49	80.55	39.58	79.15	40.62
12	83.20	37.39	81.87	38.53	80.51	39.61	79.11	40.66
14	83.15	37.43	81.83	38.56	80.64	39.65	79.06	40.69
16	83.11	37.47	81.78	38.60	80.41	39.69	79.01	40.72
18	83.07	37.51	81.74	38.64	80.37	39.72	78.96	40.76
20	83.02	37.54	81.69	38.67	80.32	39.76	78.92	40.79
22	82.98	37.58	81.65	38.71	80.28	39.79	78.87	40.82
24	82.93	37.62	81.60	38.75	80.23	39.83	78.82	40.86
26	82.89	37.66	81.56	38.78	80.18	39.86	78.77	40.89
28	82.85	37.70	81.51	38.62	80.14	39.90	78.73	40.92
30	82.80	37.74	81.47	38.86	80.09	39.93	78.68	40.96
32	82.76	37.77	81.42	38.89	80.04	39.97	78.63	40.99
34	82.72	37.81	81.38	38.93	80.00	40.00	78.58	41.02
36	82.67	37.85	81.33	38.97	79.95	40.04	78.54	41.06
38	82.63	37.89	81.28	39.00	79.90	40.07	78.49	41.09
40	82.58	37.93	81.24	39.04	79.86	40.11	78.44	41.12
42	82.54	37.96	81.19	39.08	79.81	40.14	78.38	41.16
44	82.49	38.00	81.15	39.11	79.76	40.18	78.34	41.19
46	82.45	38.04	81.10	39.15	79.72	40.21	78.30	41.22
48	82.41	38.08	81.06	39.18	79.67	40.24	78.25	41.26
50	82.36	38.11	81.01	39.22	79.62	40.28	78.20	41.29
52	82.32	38.15	80.97	39.26	79.58	40.31	78.15	41.32
54	82.27	38.19	80.92	39.29	79.53	40.35	79.10	41.35
56	82.23	38.23	80.87	39.33	79.48	40.38	79.06	41.39
58	82.18	38.26	80.83	39.36	76.44	40.42	78.01	41.42
60	82.14	38.30	80.78	39.40	79.39	40.45	77.96	41.45
(f+c)=0.75	0.68	0.31	0.68	0.32	0.67	0.33	0.66	0.35
(f+c)=1.00	0.91	0.41	0.90	0.43	0.89	0.45	0.89	0.46

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

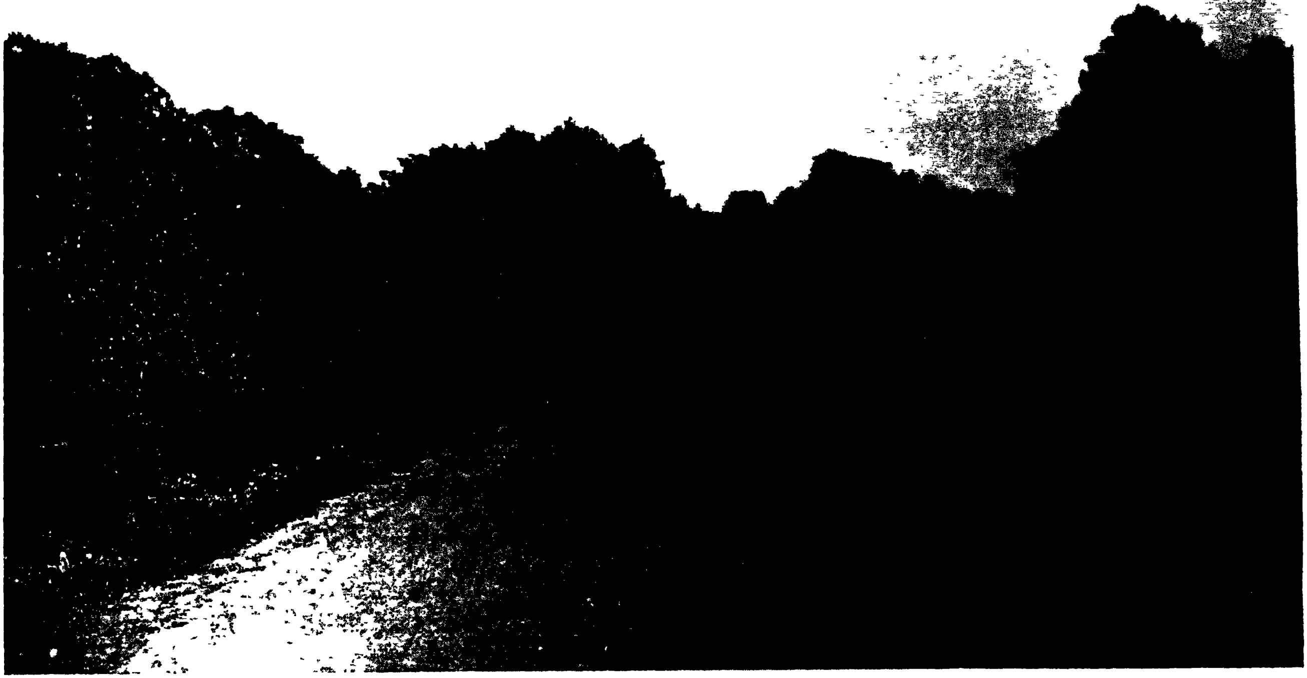
Minutes	28°		29°		30°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	77.96	41.45	76.50	42.40	75.00	43.30
2	77.91	41.48	76.45	42.43	74.95	43.33
4	77.86	41.52	76.40	42.46	74.90	43.36
6	77.81	41.55	76.35	42.49	74.85	43.39
8	77.77	41.58	76.30	42.53	74.80	43.42
10	77.72	41.61	76.25	42.56	74.75	43.45
12	77.67	41.65	76.20	42.59	74.70	43.47
14	77.62	41.68	76.15	42.62	74.65	43.50
16	77.57	41.71	76.10	42.65	74.60	43.53
18	77.52	41.74	76.05	42.68	74.55	43.56
20	77.48	41.77	76.00	42.71	74.49	43.59
22	77.42	41.81	75.95	42.74	74.44	43.62
24	77.38	41.84	75.90	42.77	74.39	43.65
26	77.33	41.87	75.85	42.80	74.34	43.67
28	77.28	41.90	75.80	42.83	74.29	43.70
30	77.23	41.93	75.75	42.86	74.24	43.73
32	77.18	41.97	75.70	42.89	74.19	43.76
34	77.13	42.00	75.65	42.92	74.14	43.79
36	77.09	42.03	75.60	42.95	74.09	43.82
38	77.04	42.06	75.55	42.98	74.04	43.84
40	76.99	42.09	75.50	43.01	73.99	43.87
42	76.94	42.12	75.45	43.04	73.93	43.90
44	76.89	42.15	75.40	43.07	73.88	43.93
46	76.84	42.19	75.35	43.10	73.83	43.95
48	76.79	42.22	75.30	43.13	73.78	43.98
50	76.74	42.25	75.25	43.16	73.73	44.01
52	76.69	42.28	75.20	43.18	73.68	44.04
54	76.64	42.31	75.15	43.21	73.63	44.07
56	76.59	42.34	75.10	43.24	73.58	44.09
58	76.55	42.37	75.05	43.27	73.52	44.12
60	76.50	42.40	75.00	43.30	73.47	44.15
(f+c)=0.75	0.66	0.36	0.65	0.37	0.65	0.38
(f+c)=1.00	0.88	0.48	0.87	0.49	0.86	0.51

TABLE II. Dimensions and Co-ordinates for Map Construction.

Length of 1° measured on a parallel in statute miles	Length of 1° measured on a meridian in statute miles	Length of side of tangent cone in statute miles	MAP CO-ORDINATES—IN METERS													
			0° 15′ from center		0° 30′ from center		0° 45′ from center		1° from center		1° 30′ from center		2° from center		3° from center	
			X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
62.729	68.829	8504	25238	23	50476	93	75713	209	100951	372	151425	838	201896	1489	302831	3351
62.212	68.839	8131	25030	24	50060	96	75089	215	100118	383	150176	862	200231	1532	300332	3447
61.676	68.848	7784	24814	25	49628	98	74442	221	99256	393	148882	885	198505	1573	297742	3539
61.122	68.858	7459	24591	25	49182	101	73772	227	98363	403	147542	907	196719	1612	295062	3627
60.548	68.869	7156	24360	26	48720	103	73080	232	97440	412	146157	928	194872	1649	292291	3710
59.956	68.879	6870	24122	26	48244	105	72366	237	96487	421	144728	947	192967	1684	289432	3789
59.345	68.890	6602	23876	27	47753	107	71629	241	95505	429	143255	966	191002	1717	286484	3863
58.716	68.901	6349	23624	27	47247	109	70871	246	94494	437	141738	983	188980	1748	283449	3933
58.071	68.912	6109	23364	28	46727	111	70091	250	93454	444	140178	999	186899	1777	280328	3997
57.407	68.923	5882	23097	28	46193	113	69289	254	92385	451	138575	1014	184762	1803	277121	4057
56.725	68.935	5666	22822	29	45645	114	68467	257	91289	457	136930	1028	182568	1828	273830	4112
56.027	68.946	5461	22541	29	45083	116	67624	260	90164	462	135243	1041	180319	1854	270455	4162

TABLE II. Dimensions and Co-ordinates for Map Construction.

Length of 1° measured on a parallel in statute miles	Length of 1° measured on a meridian in statute miles	Length of side of tangent cone in statute miles	MAP CO-ORDINATES—IN METERS													
			0° 15′ from center		0° 30′ from center		0° 45′ from center		1° from center		1° 30′ from center		2° from center		3° from center	
			X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
4.579	68.969	5079	21959	29	43917	118	65875	265	87833	472	131746	1062	175656	1888	263458	4247
3.829	68.981	4901	21657	30	43314	119	64971	268	86627	476	129937	1070	173243	1903	259839	4281
3.063	68.993	4730	21349	30	42698	120	64046	269	85394	479	128088	1078	170778	1916	256140	4311
2.281	69.006	4566	21031	30	42068	120	63102	271	84136	482	126200	1084	168260	1927	252363	4335
1.483	69.018	4408	20713	30	41426	121	62139	272	82851	484	124273	1088	165691	1935	248508	4354
0.669	69.030	4257	20386	30	40771	121	61157	273	81541	485	122308	1092	163071	1941	244578	4367
9.840	69.042	4111	20052	30	40101	121	60156	273	80206	486	120306	1094	160401	1945	240572	4375
8.995	69.054	3970	19712	30	39424	122	59136	274	78847	486	118267	1095	157682	1946	236493	4378
8.136	69.066	3834	19366	30	38733	122	58098	273	77464	486	116192	1094	154915	1945	232342	4376
7.261	69.079	3703	19015	30	38029	121	57043	273	76056	485	114080	1092	152100	1942	228119	4368
6.372	69.091	3575	18657	30	37314	121	55970	272	74626	484	111934	1089	149239	1936	223827	4355
5.469	69.103	3452	18294	30	36587	120	54880	271	73172	482	109754	1084	146331	1928	219465	4337
4.552	69.115	3332	17924	30	35849	120	53773	270	71696	479	107540	1078	143379	1917	215037	4318



EXCELLENT EXAMPLE OF COUNTRY HIGHWAY MADE DUSTLESS BY THE USE OF TARVIA SURFACE

Courtesy of Barrett Manufacturing Company, New York City, New York

HIGHWAY CONSTRUCTION

PART I

COUNTRY ROADS AND BOULEVARDS

RESISTANCE TO MOVEMENT OF VEHICLES

The object of a road is to provide a way for the transportation of persons and goods from one place to another with the least expenditure of power and expense. The facility with which this traffic or transportation may be conducted over any given road depends upon the resistance offered to the movement of vehicles. This resistance is composed of: (1) resistance offered by the roadway, which consists of (a) “friction” between the surface of the road and the wheel tires, (b) resistance offered to the rolling of the wheels occasioned by the want of uniformity in the road surface or lack of strength to resist the penetrating efforts of loaded wheels, (c) resistance due to gravity called “grade resistance”; (2) resistance offered by vehicles, termed “axle friction”; and (3) resistance of the air. The magnitude of each of the components has a wide range, varying with the kind and condition of the road and its surface, the form and condition of the vehicle, the load, and the speed.

Resistance to Traction. The combination of road resistances is designated by the general term “resistance to traction”, the magnitude of which is measured by the number of pounds of effort per ton of the load required to overcome it; this is ascertained by a form of spring-balance variously called “dynograph”, “tractograph”, etc., one end of which is attached to the vehicles and the other end to the draft animals.

The road which offers the least resistance to traffic should combine a surface on which the friction of the wheels is reduced to the least possible amount, while possessing sufficient roughness to afford good foothold for the draft animals and good adhesion for motor vehicles; and should be so located as to give the most direct route with the least gradients.

TABLE I

Resistance to Traction on Different Road Surfaces

ROAD SURFACE	TRACTION RESISTANCE	
	Pounds per Ton	In Terms of Load
Earth road—ordinary condition	50 to 200	$\frac{1}{40}$ to $\frac{1}{10}$
Gravel	50 to 100	$\frac{1}{40}$ to $\frac{1}{20}$
Sand	100 to 200	$\frac{1}{20}$ to $\frac{1}{10}$
Macadam	30 to 100	$\frac{1}{67}$ to $\frac{1}{20}$
Plank road	30 to 50	$\frac{1}{67}$ to $\frac{1}{40}$
Steel wheelway	15 to 40	$\frac{1}{133}$ to $\frac{1}{50}$

Friction. The resistance of friction arises from the rubbing of the wheel tires against the surface of the road; its amount can be determined only by experiment for each kind of road surface. From many experiments the following deductions are drawn:

(1) The resistance to traction is directly proportional to the pressure.

(2) On solid unyielding surfaces, the resistance is independent of the width of the tire; but on compressible surfaces it decreases as the width of the tire increases. There is no material advantage gained, however, in making a tire more than 4 inches wide, for the reason that it is impossible to distribute the load evenly over the road owing to the irregularities and curvatures of its surface.

(3) On uniformly smooth surfaces, the resistance is independent of the speed.

(4) On rough irregular surfaces, which give rise to constant concussion, the resistance increases with the speed.

Table I shows the relative resistance to traction of various surfaces. These coefficients refer to the power required to keep the load in motion. It requires from two to six or eight times as much force to start a load as it does to keep it in motion at two or three miles per hour. The extra force required to start a load is due in part to the fact that during the stop the wheel may settle into the road surface, in part to the fact that the axle friction at starting is greater than after motion has begun; and in part to the fact that energy is consumed in accelerating the load.

Resistance to Rolling. Resistance to rolling is caused partly by the wheel penetrating or sinking below the surface of the road, forming a depression or rut, as shown in Fig. 1, thus compelling the wheel to be continually rolling up a short incline. The measure of this resistance is the horizontal force necessary at the axle to roll it up the incline; and is equal to the product of the load multiplied by one-third of the semi-chord of the submerged arc of the wheel.

Resistance to rolling is also caused by the wheel striking or colliding with loose or projecting stones, which suddenly checks the motive power; the momentum thus destroyed varies with the height of the stone or obstacle and is often considerable.

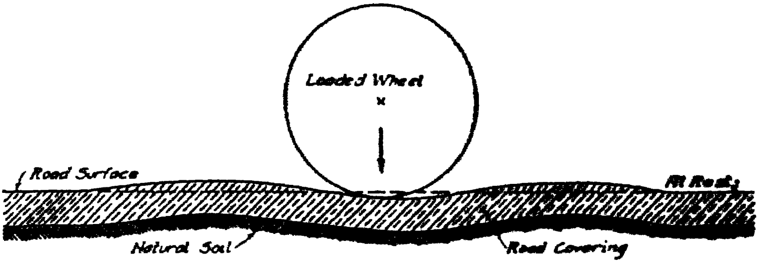


Fig. 1. Exaggerated Section of Road under Pressure of Loaded Vehicle

In both cases the power required to overcome the resistance is affected largely by the diameter of the wheel, as the larger the wheel the less force is required to lift it over the obstruction or to roll it up the inclination due to the indentation of the surface.

Illustrative Example. The power required to draw a wheel over a stone or any obstacle, such as S in Fig. 2, may be thus calculated:

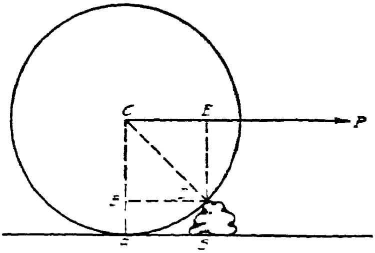


Fig. 2. Diagram for Calculating Power Required to Draw Wheel over Resisting Object

Let P represent the power sought, or that which would just balance the weight on the point of the stone, and the slightest increase of which would draw it over. This power acts in the direction CP with the leverage of BC or DE. The force of gravity W resists in the direction CB with the leverage BD. The equation of equilibrium will be $P \times CB = W \times BD$, whence

$$P = W \cdot \frac{BD}{CB} = W \cdot \frac{\sqrt{CD^2 - BC^2}}{CA - AB}$$

Let the radius of the wheel equal $CD = 26$ inches, and the height of the obstacle equal $AB = 4$ inches. Let the weight $W = 500$ pounds, of which 200 pounds may be the weight of the wheel and 300 pounds the load on the axle. The formula then becomes

$$P = 500\frac{\sqrt{676 - 484}}{26 - 4} = 500\,\frac{13.85}{22} = 314.7 \text{ lb.}$$

The pressure at the point D is compounded of the weight and the power, and equals

$$W\frac{CD}{CB} = 500 \times \frac{26}{22} = 591 \text{ lb.}$$

Therefore this pressure acts with this great effect to destroy the road in its collision with the stone; in addition there is to be considered the effect of the blow given by the wheel in descending from it. For minute accuracy the non-horizontal direction of the draft and the thickness of the axle should be taken into account. The power required is lessened by proper springs to vehicles, by enlarged wheels, and by making the line of draft ascending.

Illustrative Example. The mechanical advantage of the wheel in surmounting an obstacle may be computed from the principle of the lever. Let the wheel, Fig. 3, touch the horizontal line of traction in the point A and meet a protuberance BD. Suppose the line of draft CP to be parallel to AB. Join CD and draw the perpendiculars DE and DF. We may suppose the power to be applied at E and the weight at F, and the action is then the same as the bent lever EDF turning round the fulcrum at D. Hence $P : W :: FD : DE$. But $FD : DE :: \tan FCD : 1$; and $\tan FCD = \tan 2\, DAB$; therefore $P = W \tan 2\, DAB$. Now it is obvious that the angle DAB increases as the radius of the circle diminishes; therefore, the weight W being constant, the power required to overcome an obstacle of given height is diminished when the diameter is increased. Large wheels are, therefore, the best adapted for surmounting inequalities of the road.

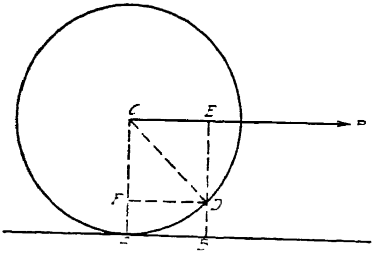


Fig. 3. Force Diagram for Wheel Drawn over Obstacle

TABLE II

Resistance Due to Gravity on Different Inclinations

Grade 1 inch	20	30	40	50	60	70	80	90	100	200	300	400
Rise in feet per mile	264	176	132	105	88	75	66	58	52	26	17	13
Resistance in pounds per ton	100	$66\frac{2}{3}$	50	40	$33\frac{1}{3}$	$28\frac{4}{7}$	25	$22\frac{2}{9}$	20	10	$6\frac{2}{3}$	5

There are, however, circumstances which provide limits to the height of the wheels of vehicles. If the radius *AC* exceeds the height of that part of the horse to which the traces are attached, the line of traction *CP* will be inclined to the horse, and part of the power will be exerted in pressing the wheel against the ground. The best average size of wheels is considered to be about 6 feet in diameter. Wheels of large diameter do less damage to a road than small ones, and cause less draft for the horses. With the same load, a two-wheeled cart does far more damage than one with four wheels, and this because of their sudden and irregular twisting motion in the trackway.

Grade Resistance. Grade resistance is due to the action of gravity, and is the same on good and bad roads. On level roads its effect is immaterial, as it acts in a direction perpendicular to the plane of the horizon and neither accelerates nor retards motion. On inclined roads it offers considerable resistance, proportional to the steepness of the incline. The resistance due to gravity on any incline in pounds per ton is equal to

$$\frac{2000}{\text{rate of grade}}$$

Table II shows the resistance due to gravity on different grades. The additional resistance caused by inclines may be investigated in the following illustrated example.

Illustrative Example. Suppose the whole weight to be borne on one pair of wheels, and that the tractive force is applied in a direction parallel to the surface of the road.

Let *AB*, Fig. 4, represent a portion of the inclined road, *C* being a vehicle just sustained in its position by a force acting in the direction *CD*. It is evident that the vehicle is kept in its position by three forces: namely, by its own weight *W* acting in the vertical to the surface of the road; and by the pressure P which the vehicle exerts against the surface of the road acting in the direction CE perpendicular to the same. To determine the relative magnitude of these three forces, draw the horizontal line AG and the vertical line BG; then, since the two lines CF and BG are parallel and are both cut by the line AB, they must make the two angles CFE and ABG equal; also the two angles CEF and AGB are equal; therefore, the remaining angles FCE and BAG are equal, and the two triangles CFE and ABG are similar. And as the three sides of the former are proportional to the three forces by which the vehicle is sustained, so also are the three sides of the latter; namely, the length of the road AB is proportional to W, or the weight of the vehicle; the vertical rise BG is proportional to F, or the force required to sustain the vehicle on the incline; and the horizontal distance AG in which the rise occurs is proportional to P, or the force with which the vehicle presses upon the surface of the road. Therefore,

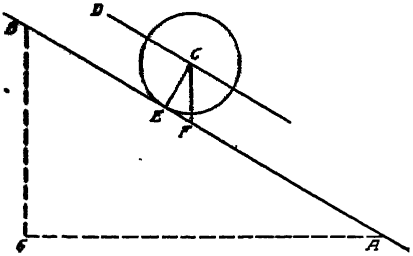


Fig. 4. Forces Acting on Vehicle When Drawn up Inclined Road

$$W : AB :: F : GB$$

and

$$W : AB :: P : AG$$

If to AG such a value be assigned that the vertical rise of the road is exactly one foot, then

$$F = \frac{W}{AB} = \frac{W}{\sqrt{AG^2+1}} = W \sin A$$

and

$$P = \frac{W \times AG}{AB} = \frac{W \times AG}{\sqrt{AG^2+1}} = W \cos A$$

in which A is the angle BAG.

To find the force requisite to sustain a vehicle upon an inclined road (the effects of friction being neglected), divide the weight of the vehicle and its load by the inclined length of the road, the vertical rise of which is one foot, and the quotient is the force required.

[illegible]

TABLE III

Tractive Power of Horses at Different Velocities

MILES PER HOUR	TRACTIVE FORCE (lb)	MILES PER HOUR	TRACTIVE FORCE (lb.)
¾	333.33	2¼	111 11
1	250	2½	100
1¼	200	2¾	90.91
1½	166 66	3	83.33
1¾	142.86	3½	71.43
2	125	4	62.50

To find the pressure of a vehicle against the surface of an inclined road, multiply the weight of the loaded vehicle by the horizontal length of the road, and divide the product by the inclined length of the same; the quotient is the pressure required. The force with which a vehicle presses upon an inclined road is always less than its actual weight; the difference is so small that, unless the inclination is very steep, it may be taken equal to the weight of the loaded vehicle.

To find the resistance to traction in passing up or down an incline, ascertain the resistance on a level road having the same surface as the incline, to which add, if the vehicle ascends, or subtract, if it descends, the force requisite to sustain it on the incline; the sum or difference, as the case may be, will express the resistance.

Tractive Power and Gradients. Although transportation by mechanically propelled vehicles will continue to increase, it is not probable that for many years it will become more important than traffic drawn by animals; and as mechanically propelled vehicles can ascend any grade feasible for animals, it is only necessary to discuss the effect of grades on horse-drawn traffic.

Tractive Power of Horses. The necessity for easy grades is dependent upon the power of the horse to overcome the resistance to motion, which is composed of four forces, viz, friction, collision, gravity, and resistance of air. All estimates on the tractive power of horses must to a certain extent be vague, owing to the different strengths and speeds of animals of the same kind, as well as to the extent of their training to any particular kind of work.

The draft or pull which a good average horse, weighing 1,200 pounds, can exert on a level, smooth road at a speed of 2½ miles per

TABLE IV

Variation of Tractive Power with Time

Hours per Day	Tractive Force (lb)	Hours per Day	Tractive Force (lb.)
10	100	7	146$\frac{6}{7}$
9	111$\frac{1}{9}$	6	166$\frac{2}{3}$
8	125	5	200

Note: The tractive power of teams may be found by multiplying the above values by the following constants:

1 horse =1
2 horses 0 95×2=1.90
3 horses 0 85×3=2.55
4 horses 0.80×4=3.20

hour is 100 pounds; which is equivalent to 22,000 foot-pounds per minute, or 13,200,000 foot-pounds per day of 10 hours. The tractive power diminishes as the speed increases and, perhaps, within certain limits, say from ¾ mile to 4 miles per hour, nearly in inverse proportion to it. Thus the average tractive force of a horse, on a level, and actually pulling for 10 hours, may be assumed approximately as shown in Table III.

The work done by a horse is greatest when the velocity with which he moves is one-eighth of the greatest velocity with which he can move when unloaded; and the force thus exerted is 0.45 of the utmost force that he can exert at a dead pull. The tractive power of a horse may be increased in about the same proportion as the time is diminished, so that when working from 5 to 10 hours on a level, it will be about as shown in Table IV.

Loss of Tractive Power on Inclines. In ascending inclines a horse's power diminishes rapidly; a large portion of his strength is expended in overcoming the resistance of gravity due to his own weight and that of the load. Table V shows that as the steepness of the grade increases, the efficiency of both the horse and the road surface diminishes; that the more of the horse's energy which is expended in overcoming gravity, the less remains to overcome the surface resistance.

Table VI shows the gross load which an average horse, weighing 1,200 pounds, can draw on different kinds of road surfaces, on a level and on grades rising 5 and 10 feet per 100 feet.

TABLE V

Effects of Grades upon the Load a Horse Can Draw on Different Pavements

GRADE	EARTH	BROKEN STONE	STONE BLOCKS	ASPHALT
Level	1.00	1.00	1.00	1.00
1 : 100	.80	.66	.72	.41
2 : 100	.66	.50	.55	.25
3 : 100	.55	.40	.44	.18
4 : 100	.47	.33	.36	.13
5 : 100	.41	.29	.30	.10
10 : 100	.26	.16	.14	.04
15 : 100	.10	.05	.07	..
20 : 100	.04	...	.03	...

The decrease in the load which a horse can draw upon an incline is not due alone to gravity; it varies with the amount of foothold afforded by the road surface. The tangent of the angle of inclination should not be greater than the coefficient of tractional resistance. Therefore, it is evident that the smoother the road surface, the easier should be the grade; the smoother the surface the less the foothold, and consequently the less the possible load.

The loss of tractive power on inclines is greater than any investigation will show; for, besides the increase of draft caused by gravity, the power of the horse is much diminished by fatigue upon a long ascent, and even in greater ratio than man, owing to its anatomical formation and great weight. Though a horse on a level is as strong as five men, on a grade of 15 per cent, it is less strong than three; for three men carrying each 100 pounds will ascend such a grade faster and with less fatigue than a horse with 300 pounds.

A horse can exert for a short time twice the average tractive pull which he can exert continuously throughout the day's work; hence, so long as the resistance on the incline is not more than double the resistance on the level, the horse will be able to take up the full load which he is capable of drawing.

Steep grades are thus seen to be objectionable, and particularly so when a single one occurs on an otherwise comparatively level road, in which case the load carried over the less inclined portions must be reduced to what can be hauled up the steeper portion.

The bad effects of steep grades are especially felt in winter.

TABLE VI

Gross Loads for Horse on Different Pavements on Different Grades

Description of Surface	Level	Grade (5 per cent)	Grade (10 per cent)
Asphalt	13,216	.....	.....
Broken stone (best condition)	6,700	1,840	1,060
Broken stone (slightly muddy)	4,700	1,500	1,000
Broken stone (ruts and mud)	3,000	1,390	890
Broken stone (very bad condition)	1,840	1,040	740
Earth (best condition)	3,600	1,500	930
Earth (average condition)	1,400	900	660
Earth (moist but not muddy)	1,100	780	600
Stone-block pavement (dry and clean)	8,300	1,920	1,090
Stone-block pavement (muddy)	6,250	1,800	1,040
Sand (wet)	1,500	675	390
Sand (dry)	1,087	445	217

when ice covers the roads, for the slippery condition of the surface causes danger in descending, as well as increased labor in ascending; during heavy rains the water also runs down the road and gulleys it out, destroying its surface, thus causing a constant expense for repairs. The inclined portions are subject to greater wear from horses ascending, thus requiring thicker covering than the more level portions, and hence increasing the cost of construction.

It will rarely be possible, except in a flat or comparatively level country, to combine easy grades with the best and most direct route. These two requirements will often conflict. In such a case, increase the length of the road. The proportion of this increase will depend upon the friction of the covering which is adopted. But no general rule can be given to meet all cases as respects the length which may thus be added, for the comparative time occupied in making the journey forms an important element in any case which arises for settlement. Disregarding time, the horizontal length of a road may be increased to avoid a 5 per cent grade, seventy times the height.

Table VII shows, for most practical purposes, the force required to draw loaded vehicles over inclined roads. In the fifth column the length given is the length which would require the same motive power to be expended in drawing the load over it, as would be necessary to draw over a mile of the inclined road. The loads given in the sixth column are the maximum loads which average

TABLE VII

Data for Loaded Vehicles over Inclined Roads

Rate of Grade (ft. per 100 ft)	Pressure on Plane (lb per ton)	Tendency down Plane (lb. per ton)	Power Required to Haul 1 Ton up Plane (lb)	Equivalent Length of Level Road (mi)	Maximum Load Horse Can Haul (lb.)
0.00	2240	.00	45 00	1.000	6270
0.25	2240	5.60	50 60	1.121	5376
0.50	2240	11.20	56 20	1 242	4973
0.75	2240	16 80	61 80	1.373	4490
1.00	2240	22.40	67 40	1.500	4145
1.25	*2240	28 00	73 00	1.622	3830
1.50	2240	33.60	78 60	1.746	3584
1.75	2240	39.20	84 20	1 871	3290
2.00	2240	45.00	90 00	2.000	3114
2 25	2240	50 40	95 40	2.120	2935
2.50	2240	56.00	101.00	2.244	2725
2 75	2240	61 33	106.33	2.363	2620
3.00	2239	67.20	112.20	2.484	2486
4 00	2238	89.20	134 20	2.982	2083
5.00	2237	112 00	157.00	3 444	1800
6 00	2233	134.40	179.40	3 986	1568
7 00	2232	156 80	201.80	4.844	1367
8 00	2232	179 20	224.20	4 982	1235
9 00	2231	201 60	246 60	4.840	1125
10 00	2229	224 00	269 00	5 977	1030

* Near enough for practice; actually 2239 888

Pressure on plane = weight × nat cos of angle of plane

horses weighing 1,200 pounds can draw over such inclines, the friction of the surface being taken at $\frac{1}{50}$ of the load drawn.

Axle Friction. The resistance of the hub to turning on the axle is the same as that of a journal revolving in its bearing, and has nothing to do with the condition of the road surface. The coefficient of journal friction varies with the material of the journal and its bearing, and with the lubricant. It is nearly independent of the velocity, and seems to vary about inversely as the square root of the pressure. For light carriages when loaded, the coefficient of friction is about 0.020 of the weight on the axle; for the ordinary thimble-skein wagon when loaded, it is about 0.012. These coefficients are for good lubrication; if the lubrication is deficient, the axle friction is 2 to 6 times as much as above.

The tractive power required to overcome the above axle friction for carriages of the usual proportions is about 3 to $3\frac{3}{4}$ pounds per ton of the weight on the axle; and for truck wagons, which have medium sized wheels and axles, it is about $3\frac{1}{2}$ to $4\frac{1}{2}$ pounds per ton.

TABLE VIII

Wind Pressure for Various Vehicles

DESCRIPTION	VELOCITY OF WIND (mi per hour)	WIND PRESSURE (lb per sq ft)
Pleasant breeze	15	1.107
Brisk gale	20	1.968
	25	3.075
High wind	30	4.428
	35	6.027
Very high wind	40	7.782
	45	9.963
Storm	50	12.300

Effect of Springs on Vehicles. Experiments have shown that springs mounted in vehicles materially decrease the resistance to traction and diminish the effects caused in the vertical plane by irregularities of the surface; but they do not diminish the horizontal component which is the one that causes the greatest wear of the road, especially at speeds beyond a walking pace. The vehicles with springs were found not to cause more wear with the horses going at a trot than vehicles without springs when the horses were walking, all other conditions being similar. Vehicles with springs improperly fixed cause considerable concussion which, in turn, destroys the road covering.

Resistance of Air. The resistance arising from the force of the wind will vary with the velocity of the wind, with the velocity of the vehicle, with the area of the surface acted upon, and also with the angle of incidence of direction of the wind with the plane of the surface. Table VIII gives the force per square foot for various velocities.

LOCATION OF ROADS

The considerations governing the location of roads are dependent upon the commercial condition of the country to be traversed. In old and long-inhabited sections, the controlling element will be the character of the traffic to be accommodated. In such a section, the route is generally predetermined and, therefore, there is less liberty of choice and selection than in a new and sparsely settled district, where the object is to establish the easiest, shortest, and most economical line of intercommunication according to the physical character of the ground.

Whichever of these two cases may have to be dealt with, the same principle governs the engineer, namely, so to lay out the road as to effect the conveyance of the traffic with the least expenditure of motive power consistent with economy of construction and maintenance.

Economy of motive power is promoted by easy grades, by the avoidance of all unnecessary ascents and descents, and by a direct line; but directness must be sacrificed to secure easy grades and to avoid expensive construction.

RECONNOISSANCE

Object of Reconnoissance. The selection of the best route demands much care and consideration on the part of the engineer. To obtain the requisite data upon which to form his judgment, he must make a personal reconnoissance of the district. This requires that the proposed route be either ridden or walked over and a careful examination made of the principal physical contours and natural features of the district. The amount of care demanded and the difficulties attending the operations will altogether depend upon the character of the country. The immediate object of the reconnoissance is to select one or more trial lines, from which the final route may be ultimately determined. When there are no maps of the section traversed, or when those which can be procured are indefinite or inaccurate, the work of reconnoitering will be much increased.

Points to Consider. In making a reconnoissance there are several points which, if carefully attended to, will very considerably lessen the labor and time otherwise required. Lines which would run along the immediate bank of a large stream must of necessity intersect all the tributaries confluent on that bank, thereby demanding a corresponding number of bridges. Those, again, which are situated along the slopes of hills are more liable in rainy weather to suffer from the washing away of the earthwork and the sliding of the embankments; the others which are placed in valleys or on elevated plateaux, when the line crosses the ridges dividing the principal watercourses, will have steep ascents and descents.

In making an examination of a tract of country, the first point to attract notice is the unevenness or undulation of its surface, which appears to be entirely without system, order, or arrangement; but upon closer examination it will be perceived that one general principle of configuration obtains even in the most irregular countries. The country is intersected in various directions by main watercourses or rivers, which increase in size as they approach the point of their discharge. Towards these main rivers lesser rivers approach on both sides, running right and left through the country, and into these, again, enter still smaller streams and brooks. The streams thus divide the hills into branches or spurs having approximately the same direction as themselves, and the ground falls in every direction from the main chain of hills towards the watercourses, forming ridges more or less elevated.

The main ridge is cut down at the heads of the streams into depressions called gaps or passes; the more elevated points are called peaks. The water which has fallen upon these peaks is the origin of the streams which have hollowed out the valleys. Furthermore, the ground falls in every direction towards the natural watercourses, forming ridges more or less elevated running between them and separating from each other the districts drained by the streams.

The natural watercourses mark not only the lowest lines, but the lines of the greatest longitudinal slope in the valleys through which they flow. The direction and position of the principal streams give also the direction and approximate position of the high ground or ridges which lie between them. The positions of the tributaries to the larger stream generally indicate the points of greatest depression in the summits of the ridges and, therefore, the points at which lateral communication across the high ground separating contiguous valleys can be most readily made.

Instruments Used. The instruments employed in reconnoitering are: the *compass*, which is used to ascertain direction; the *aneroid barometer*, to fix the approximate elevation of summits, etc.; and the *hand level*, to ascertain the elevation of neighboring points. If a vehicle can be used, an odometer may be added, but distances can usually be guessed or ascertained by time estimates closely enough for preliminary purposes. The best maps obtainable and traveling companions who possess a local knowledge of the country, together with the above outfit, are all that will be necessary for the first inspection.

PRELIMINARY SURVEY

The routes selected through the reconnoissance are examined in detail by a survey called a "preliminary survey" from the results of which the exact location can be determined.

Features to Be Considered. In making the preliminary survey, the topographical features are noted to the right and left of the transit line for a convenient distance. The data required for drawing the topography are obtained by levels taken with a leveling instrument or with a transit provided with stadia wires, on lines perpendicular to the transit line of the survey. The location of buildings, fences, streams, roads, railroads, and other objects, is determined by measurements made with a tape on lines perpendicular to the survey line; or, when the distance to the object required is considerable, the location is found by angles measured from two stations on the transit line and the distance is measured by stadia. The following information is also noted: the importance, magnitude, and direction of the streams crossed; the character of the material to be excavated or available for embankments; the position of quarries; the mode of access thereto, and the kind of stone; the position of unloading points on railroads; and any other information that might affect a selection.

Topography. *Levels.* Levels should be taken along the course of each line, usually at every 100 feet, or at closer intervals, depending upon the nature of the country. In taking the levels, the heights of all existing roads, railroads, rivers, or canals should be noted. "Bench marks" should be established at least every half mile, that is, marks made on any fixed object, such as a gate post, side of a house, or, in the absence of these, a cut made on a large tree. The height and exact description of each bench mark should be recorded in the level book.

Cross Levels. Wherever considered necessary, levels at right angles to the center line should be taken. These will be found useful in showing what effect a deviation to the right or left of the surveyed line would have. Cross levels should be taken at the

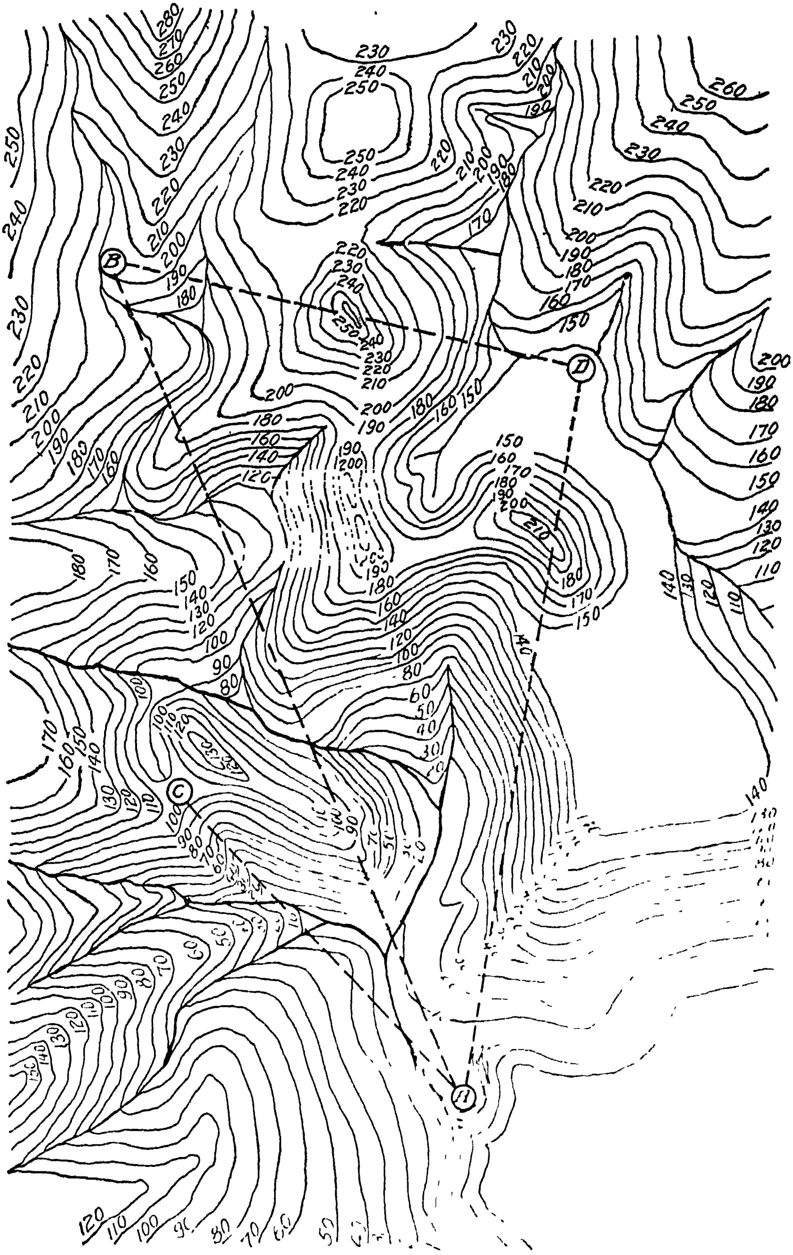


Fig. 5. Contour Map Used in Road Surveys

any, these levels will have to be altered to suit the levels of the proposed road.

Contours. The levels of the transit and cross lines are worked into a map that shows the irregularities of the ground with reference to its elevations and depressions. Various methods are employed for delineating these upon paper. For the purpose of the engineer the method of contours, Fig. 5, is the most serviceable, since by it the true shape of the hills and valleys can be shown.

Contours are lines drawn through the points of equal altitude; that is, every point of the ground over which a contour line passes is at a certain height above a known fixed plane called the "datum".

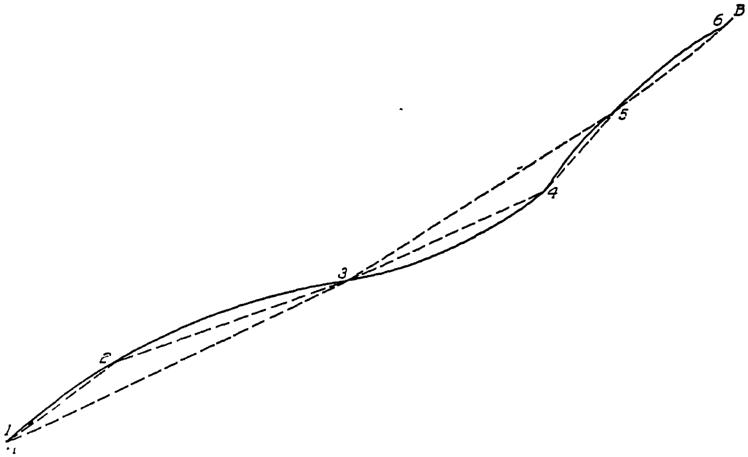


Fig. 6. Diagram Showing Method of [illegible] Elevation of Successive Contours of [illegible]

Mean sea level is the datum plane universally employed; when this cannot be conveniently used, an arbitrary plane is adopted which is below the lowest point in the territory under survey.

The difference of elevation between adjacent contour lines is called the "contour interval"; this may be one, five, ten, or more, feet. Whatever the difference adopted, it must be constant for all contours on the same map. Contours are designated by their height, expressed in feet, above the datum plane. The elevation of each contour is shown in figures at points close enough together to allow the eye to run from one to the other with ease. It is best [illegible] written alongside, the numbers should be placed on the higher side of the contour.

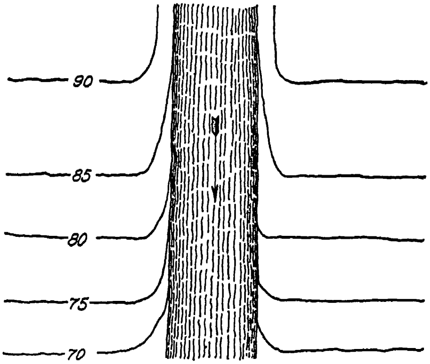


Fig. 7. Method of Showing Contours of Banks of Streams

The theory of contours is given in order that no error will be made by supposing the slope of the ground from a point in one contour to a point in the next, to be a straight line. The less the contour interval, the less error will be made. If in Fig. 6 the curved line *AB* represents the actual surface of the ground, and points *1, 3, 5,* the elevation of successive contours, the broken line *1, 3, 5* will represent the assumed ground surface, and its departure from the line *AB* is the error introduced. If now the points *2, 4, 6* are also determined, or the contour intervals be reduced one-half, the assumed slope is *1, 2, 3, 4, 5, 6,* which differs less from the line *AB* than the line *1, 3, 5,* and hence introduces less error. With points determined at very short intervals, the error is practically eliminated.

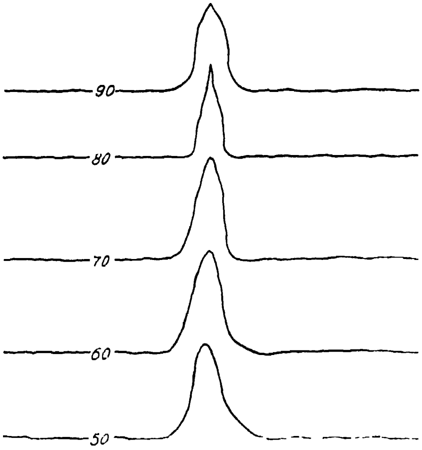


Fig. 8. Method of Showing Contour of Small Stream or Dry River Bed

A knowledge of the shape of the ground is obtained from the distance of the contours from one another. The steeper the slopes, the closer will the contours be. If in a hill the upper contours, near the summit, are closer together than those near the bottom, the intervening ground is concave; if the lower contours are closer than the higher ones, the intervening ground is convex.

Every contour must close upon itself in a loop or else must extend unbroken from one point on the margin of the map to some other point on the margin. An exception is made in the case of large streams, the contour on each bank being carried up-stream until it cuts the water surface, when it is dropped, as shown in Fig. 7. In a small stream or dry bed, the contour crosses at the point where the elevation of the bed is that of the contour, as shown in Fig. 8.

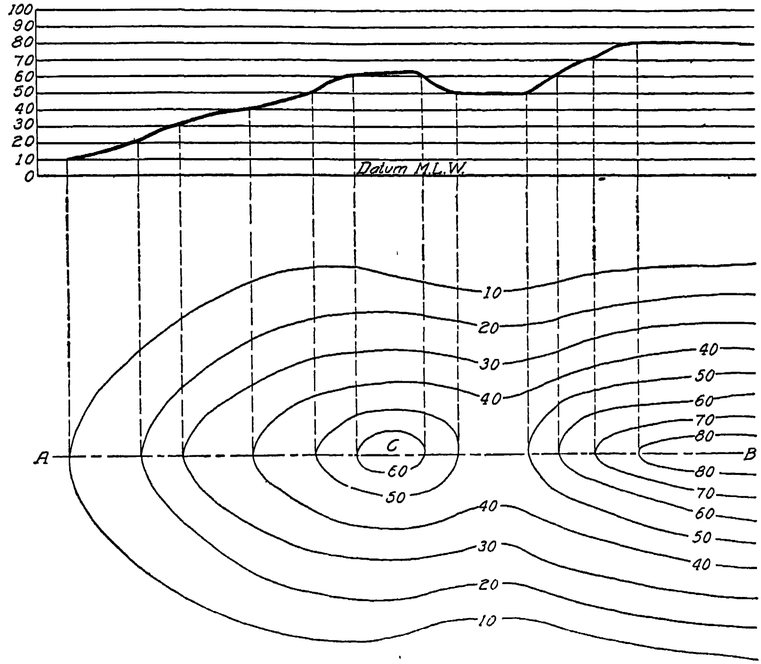


Fig. 9 Typical Profile as Obtained from Contour Map

Profile. A profile is a longitudinal section of the route. The profile in any given direction is easily made from the contour map in the manner shown in Fig. 9. Assuming that a profile is required along the line *AB*, the contours show that the ground rises from *A* to *B*, and also that a small isolated elevation occurs at *C*. The short distance between the contours near *B* indicates that the rise is steep. To obtain the profile, draw parallel lines at a distance apart equal to the vertical interval between the contours on any

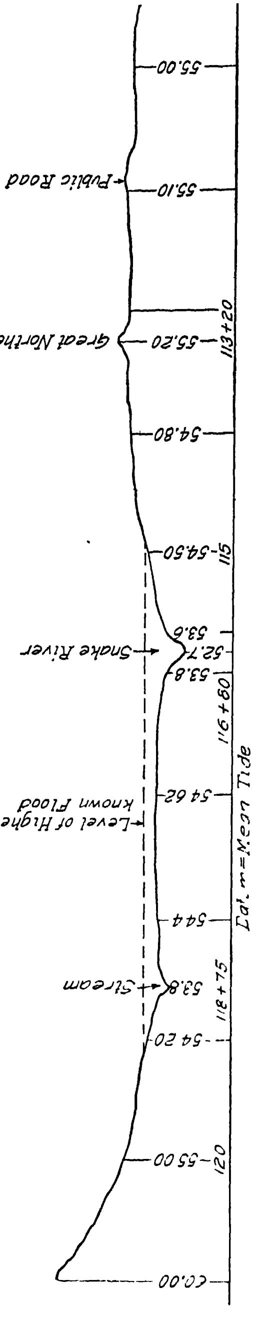


Fig 10. Typical Preliminary Profile

with the numbers on the contours. From each point on the line *AB*, where it intersects a contour, draw vertical lines to intersect the corresponding horizontal line. Connect the several points thus found, remembering the distinction between convex and concave surfaces. The profile thus obtained gives the relative heights of different points in the line *AB*, but it does not give the true gradient. The true gradient cannot be represented accurately unless the vertical intervals are drawn on the same scale as the horizontal scale of the map. If this is done, the elevations will generally be so minute that the profile will not give a sufficiently striking representation of the surface features. It is, therefore, necessary to exaggerate the vertical scale in a certain fixed proportion. A convenient scale is 400 feet horizontal and 40 feet vertical. A typical preliminary profile, with all the information which it is supposed to give, is shown in Fig. 10.

Map. The map, Fig. 11, should show the lengths and direction of the different portions of the line, the topography, rivers, water-courses, roads, railroads, and other matters of interest, such as town and county lines, dividing lines between property, timbered and cultivated lands, etc. Any convenient scale may be adopted; 100 feet to an inch will be found the most useful.

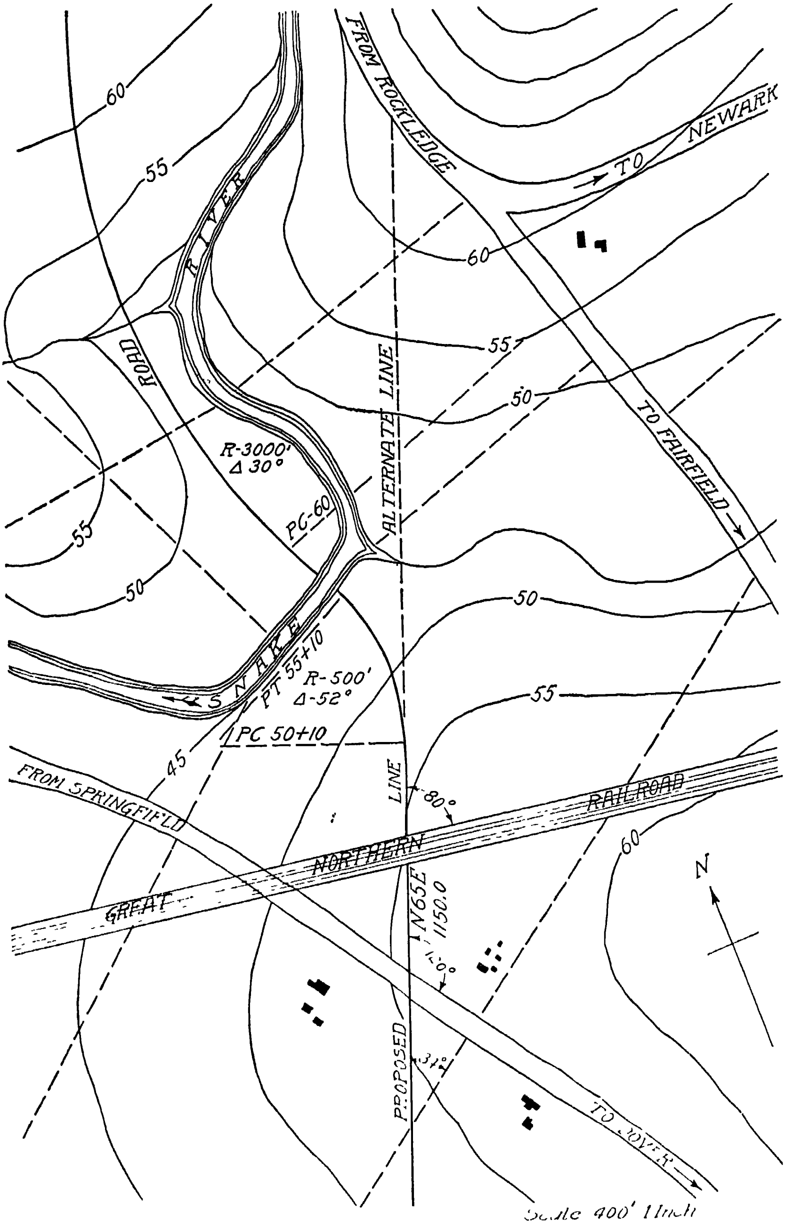


Fig. 11. Typical Map Showing Layout in the Region of the Proposed Road

Memoir. The descriptive memoir should give with minuteness all information, such as the nature of the soil, character of the several excavations whether earth or rock, and such particular features as cannot be clearly shown upon the map or profile. Special information should be given regarding the rivers crossed, as to their width, depth at highest known flood, velocity of current, character of banks and bottom, and their angle of skew with the line of road.

Bridge Sites. The question of choosing the site of bridges is an important one. If the selection is not restricted to a particular point, the river should be examined for a considerable distance above and below what would be the most convenient point for crossing; and if a better site is found, the line of the road must be made subordinate to it. If several practicable crossings exist, they must be carefully compared in order to select the one most advantageous. The following are controlling conditions: (1) Good character of river bed, affording a firm foundation. If rock is present near the surface of the river bed, the foundation will be easy of execution, and stability and economy will be insured. (2) Stability of river banks, thus securing a permanent concentration of the waters in the same bed. (3) Axis of bridge at right angles to direction of current. (4) Bends in rivers, not being suitable localities, to be avoided if possible. A straight reach above the bridge should be secured if possible.

FINAL SELECTION OF ROUTE

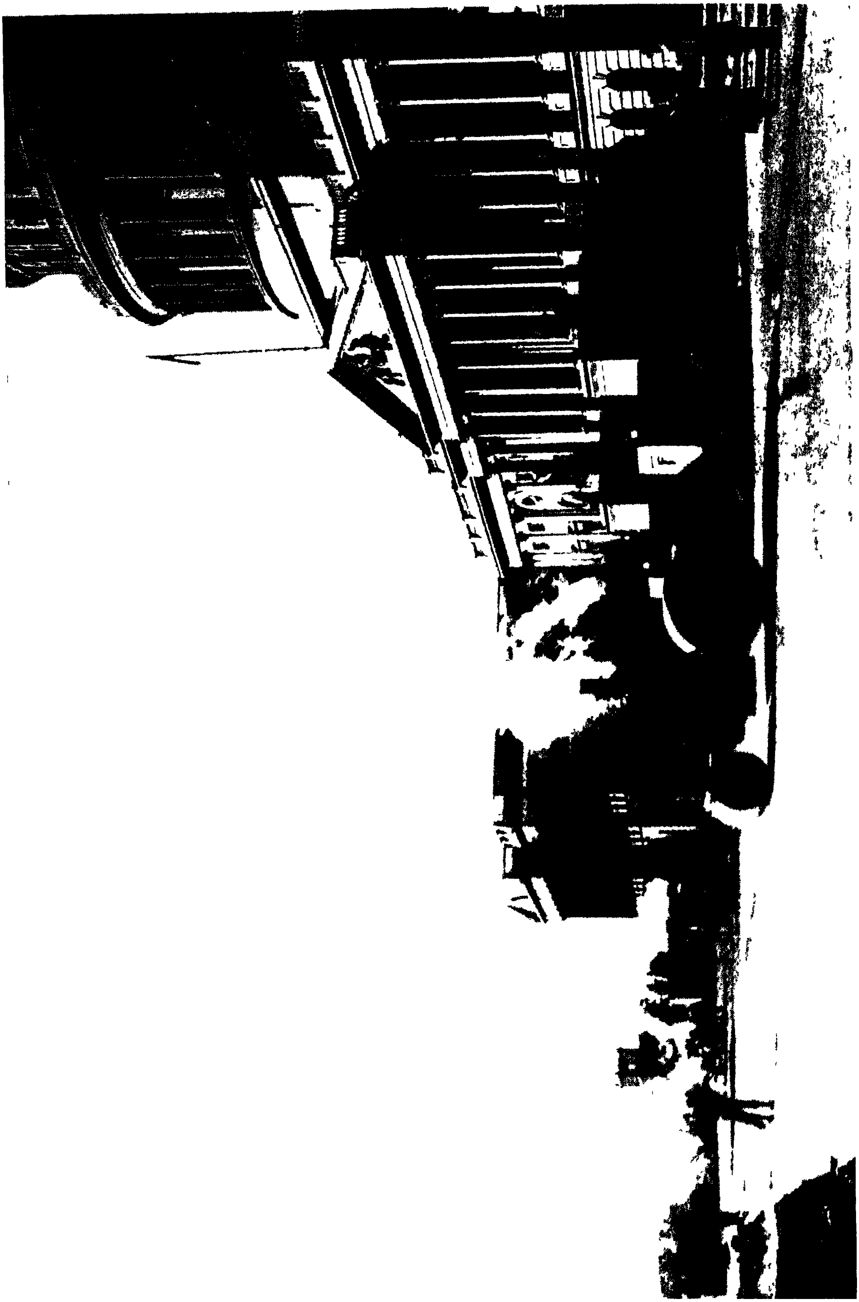
Elements Entering into Choice. In making the final selection, the following principles should be observed as far as practicable:

(1) To follow that route which affords the easiest grades. The easiest grade for a given road will depend on the kind of covering adopted for its surface.

(2) To connect the places by the shortest and most direct route commensurate with easy grades.

(3) To avoid all unnecessary ascents and descents. When a road is encumbered with useless ascents, the wasteful expenditure of power is considerable.

(4) To give the center line such a position, with reference to the natural surface of the ground, that the cost of construction shall be reduced to the smallest possible amount.



(5) To cross all obstacles, where structures are necessary, as nearly as possible at right angles. The cost of skew structures increases nearly as the square of the secant of the obliquity.

(6) To cross ridges through the lowest pass which occurs.

(7) To cross either under or over railroads; for grade crossings mean danger to every user of the highway.

Treatment of Typical Cases

Connecting Two Towns in Same or Adjacent Valleys. In laying out the line of a road, there are three cases which may have to be treated, and each of these is exemplified in the contour map, Fig. 5. *First,* the two places to be connected, as the towns *A* and *B* on the plan, may both be situated in the same valley, and upon the same side of it; that is, they are not separated from each other by the main stream which drains the valley. This is the simplest case. *Second,* although both in the same valley, the two places may be on opposite sides of the valley, as at *A* and *C*, being separated by the main river. *Third,* the two places may be situated in different valleys, separated by an intervening ridge of ground more or less elevated, as at *A* and *D*. In laying out an extensive line of road, it frequently happens that all these cases have to be dealt with. The most perfect road is that of which the course is perfectly straight and the surface practically level; and, all other things being the same, the best road is that which answers nearest to this description.

Case 1. Now, in the first case, that of the two towns situated on the same side of the main valley, there are two methods which may be pursued in forming a communication between them. A road following the direct line between them, shown by the thick dotted line *AB*, may be made, or a line may be adopted which will gradually and equally incline from one town to another, supposing them to be at different levels; or, if they are on the same level, the line should keep at that level throughout its entire course, following all the sinuosities and curves which the irregular formation of the country may render necessary for the fulfillment of these conditions. According to the first method, a level or uniformly inclined road might be made from one to the other; this line would cross all the valleys and streams which run down to the main river, thus necessitating deep cuttings, heavy embankments, and numerous bridges; or these expensive works might be avoided by following the sinuosities of the valley. When the sides of the main valley are pierced by numerous ravines with projecting spurs and ridges intervening, instead of following the sinuosities, it will be found better to make a nearly straight line cutting through the projecting points in such a way that the material excavated should be just sufficient to fill the hollows.

Of all these, the best is the straight uniformly inclined or level road, although at the same time it is the most expensive. If the importance of the traffic passing between the places is not sufficient to warrant so great an outlay, it will become a matter of consideration whether the course of the road should be kept straight, its surface being made to undulate with the natural face of the country; or whether, a level or equally inclined line being adopted, the course of the road should be made to deviate from the direct line and follow the winding course which such a condition is supposed to necessitate.

Case 2. In the second case, that of two places situated on opposite sides of the same valley, there is, in like manner, the choice of a perfectly straight line to connect them, which would probably require a big embankment if the road were kept level; or steep inclines if it followed the surface of the country; or by winding the road, it might be carried across the valley at a higher point, where, if the level road be taken, the embankment would not be so high, or, if kept on the surface, the inclination would be reduced.

Case 3. In the third case, there is, in like manner, the alternative of carrying the road across the intervening ridge in a perfectly straight line, or of deviating it to the right and left, and crossing the ridge at a point where the elevation is less. The proper determination of the question which of these courses is the best under certain circumstances involves a consideration of the comparative advantages and disadvantages of inclines and curves. What additional increase in the length of the road would be equivalent to a given inclined plane upon it; or conversely, what inclination might be given to a road as an equivalent to a given decrease in its length? To satisfy this question, the comparative force required to draw different vehicles with given loads must be known, both upon level and variously inclined roads.

The route which will give the most general satisfaction consists in following the valleys as much as possible and rising afterward by gentle grades. This course traverses the cultivated lands, regions studded with farmhouses and factories. The value of such a line is much more considerable than that of a route by the ridges. The watercourses which flow down to the main valley are, it is true, crossed where they are the largest and require works of large dimensions, but also they are fewer in number.

Treatment of Intermediate Towns. Suppose that it is desired to construct a road between two distant towns, *A* and *B*, Fig. 12, and let us for the present neglect altogether the consideration of the physical features of the intervening country, assuming that it is equally favorable whichever line we select. Now at first sight it would appear that, under such circumstances, a perfectly straight line drawn from one town to the other would be the best that could be chosen. On more careful examination, however, of the locality, we may find that there is a third town *C*, situated somewhat on one side of the straight line which we have drawn from *A* to *B*; and although our primary object is to connect only the two latter, it would nevertheless be of considerable service if all three towns were put into mutual connection with each other.

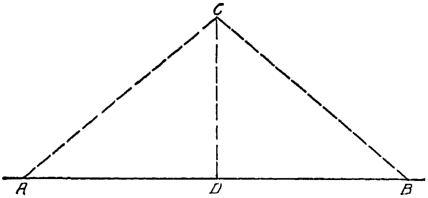


Fig 12. Diagram Showing Method of Determining Line of Road between Successive Towns

This may be effected in three different ways, any one of which might, under the circumstances, be the best. In the first place, we might, as originally suggested, form a straight road from *A* to *B*, and in a similar manner two other straight roads from *A* to *C*, and from *B* to *C*, and this would be the most perfect way of effecting the object in view, the distance between any of the two towns being reduced to the least distance possible. It would, however, be attended with considerable expense, and it would be necessary to construct a much greater length of road than according to the second plan, which would be to form, as before, a straight road from *A* to *B*, and from *C* to construct a road which should join the former

A or *B* and *C* would proceed to the point *D* and then turn off to *C*. With this arrangement, while the length of the roads would be very materially decreased, only a slight increase would be occasioned in the distance between *C* and the other two towns. The third method would be to form only the two roads *AC* and *CB*, in which case the distance between *A* and *B* would be somewhat increased, while that between *A* and *C* or *B* and *C* would be diminished, and the total length of road to be constructed also would be lessened.

As a general rule it may be taken that the last of these methods is the best and most convenient for the public; that is to say, that if the physical character of the country does not determine the course of the road, it generally will be found best not to adopt a perfectly straight line, but to vary the line so as to pass through all the principal towns near its general course.

Treatment of Mountain Roads. The location of roads in mountainous countries presents greater difficulties than in an ordinary undulating country; the same latitude in adopting undulating grades and choice of position is not permissible, for the maximum must be kept before the eye perpetually. A mountain road has to be constructed on the maximum grade or at grades closely approximating it, and but one fixed point can be obtained before commencing the survey, and that is the lowest pass in the mountain range; from this point the survey must be commenced. The reason for this is that the lower slopes of the mountain are flatter than those at their summit; they cover a larger area; and they merge into the valley in diverse undulations. Consequently, a road at the foot of a mountain may be carried at will in the desired direction by more than one route, while at the top of a mountain range any deviation from the lowest pass involves increased length of line. The engineer having less command of the ground, owing to the reduced area he has to deal with and the greater abruptness of the slopes, is liable to be frustrated in his attempt to get his line carried in the desired direction.

It is a common practice to run a mountain line up hill, but this should be avoided. Whenever an acute-angled zigzag is met with on a mountain road near the summit, the inference to be drawn is that the line, being carried up hill, on reaching the summit was too low and the zigzag was [illegible]

The only remedy in such a case is a resurvey beginning at the summit and running down hill. This method requires a reversal of that usually adopted. The grade line is first staked out and its horizontal location surveyed afterwards. The most appropriate instrument for this work is a transit with a vertical circle on which the telescope may be set to the angle of the maximum grade.

Loss of Height. "Loss of height" is to be carefully avoided in a mountain road. By loss of height is meant an intermediate rise in a descending grade. If a descending grade is interrupted by the introduction of an unnecessary ascent, the length of the road will be increased, over that due to the continuous grade, by the length of the portion of the road intervening between the summit of the rise and the point in the road on a level with that rise—a length which is double that due on the gradient to the height of the rise. For example, if a road descending a mountain rises, at some intermediate point, to cross over a ridge or spur, and the height ascended amounts to 110 feet before the descent is continued, such a road would be just one mile longer than if the descent had been uninterrupted; for 110 feet is the rise due to a half-mile length at a slope of 1 : 24.

Water on Mountain Roads. Water is needed by the workmen and during the construction of the road. It is also very necessary for the traffic, especially during hot weather; and if the road exceeds 5 miles in length, provision should be made to have the water either close to or within easy reach of the road. With a little ingenuity the water from springs above the road, if such exist, can be led down to drinking fountains for men, and to troughs for animals.

In a tropical country it would be a matter for serious consideration if the best line for a mountain road 10 miles in length or upwards, but without water, should not be abandoned in favor of a worse line with a water supply available.

Halting Places. On long lines of mountain roads, halting places should be provided at frequent intervals.

Alignment of Roads. No rule can be laid down for the alignment of a road—it will depend upon both the character of its traffic and the "lay of the land". To promote economy of transportation, it should be straight; but, if straightness is obtained at the expense increase in length, it will prove very expensive to the community that uses it.

The curving road around a hill often may be no longer than the straight one over it, for the latter is straight only with reference to the horizontal plane, while it is curved as to the vertical plane; the former is curved as to the horizontal plane, but straight as to the vertical plane. Both lines curve, and we call the one passing over the hill straight only because its vertical curvature is less apparent to our eyes. Excessive crookedness of alignment is to be avoided, for any unnecessary length causes a constant threefold waste: *first,* of the interest of the capital expended in making that unnecessary portion; *second,* of the ever recurring expense of repairing it; and *third,* of the time and labor employed in traveling over it.

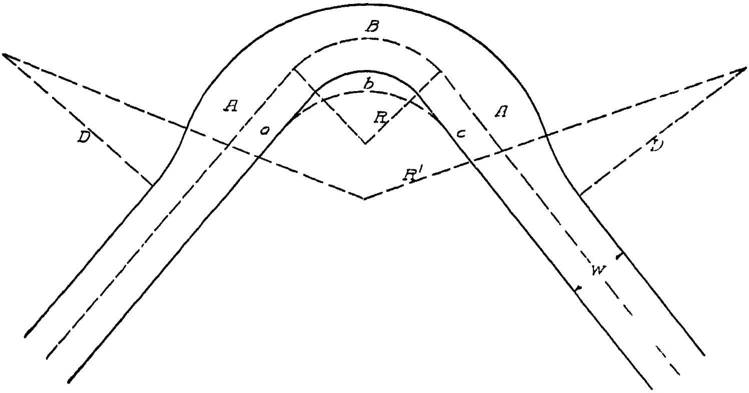


Fig. 13. Diagram Showing Method of Laying Out Curve in Road

Location and Construction of Curves. Curves, on a road used exclusively by horse-drawn traffic, should have a center radius of not less than 50 feet. On roads used by both horse-drawn and motor-vehicle traffic, the greatest possible radius should be employed and not less than 150 feet at the inner margin. Curves should not be placed at the foot of a steep ascent; and, when they occur on an ascent, the grade at that point should be decreased in order to compensate for the additional resistance of the curve.

Curves may be either circular or parabolic in form. The latter will be found exceedingly useful for joining tangents of unequal length and for following contours; when the curvature is least are less strongly marked than by a circular arc. The connection between a circular curve and its tangents should be made by a parabolic arc.

The width of the wheelway on curves should be greater than on tangents; the position in which the additional width will be of the greatest service to the traffic is at the entry arcs, as shown at *A* and *A*, Fig. 13, and not at the center *B* of the curve, which is the point commonly widened. The minimum radius of the outer curve to provide the increased width may be determined by the formula

$$R' = \sqrt{R + \left[\left(\frac{W+w}{\frac{1}{2}}\right)\right]^2 + l^2}$$

in which R is radius of inner curve; W is width of road on tangents; w is width of vehicles; and l is maximum length of vehicles, including teams. If the traffic requires it, a further widening may be obtained by flattening the inner curve as indicated by *abc*. The radius of the reversing curves should be not less than 15 feet.

The outer half of the wheelway on curves used by fast motor-vehicle traffic should be raised, as shown in Fig. 14, the amount of elevation being 4 inches for a curve of 150-foot radius and decreasing to 2 inches for a curve of 300-foot radius.

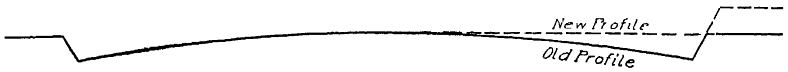


Fig. 14 Diagram Showing Adjustment of Profile on Curves for Rapidly Moving Motor Vehicles

The approach to curves should afford an unobstructed view for at least 300 feet, and obstructions which prevent the entire length of the curve from being seen by approaching vehicles should be removed.

Zigzags. The method of surmounting a height by a series of zigzags or by a series of reaches with practicable curves at the turns, is objectionable for the following reasons:

(1) An acute-angled zigzag obliges the traffic to reverse its direction without affording it convenient room for the purpose. The consequence is that with slow traffic a single train of vehicles is brought to a stand, while if two trains of vehicles, traveling in

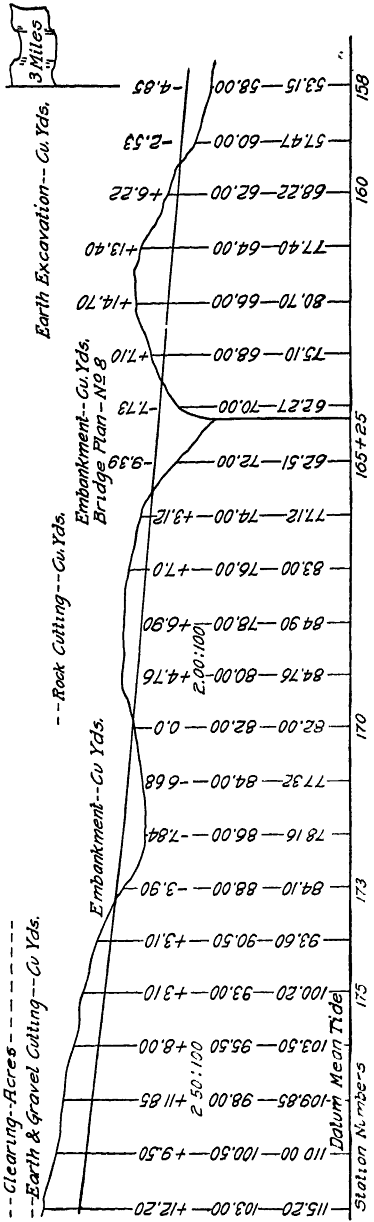


Fig 15. Typical Construction Profile

(2) With zigzags little progress is made toward the ultimate destination of the road; height is surmounted, but horizontal distance is increased without compensation.

(3) Zigzags are dangerous. In case of a runaway down hill, the zigzag must prove fatal.

(4) If the drainage cannot be carried clear of the road at the end of each reach, it must be carried under the road in one reach, only to appear again at the next, when a second bridge, culvert, or drain will be required, and so on at the other reaches. If the drainage can be carried clear at the termination of each reach, the lengths between the curves will be very short, entailing numerous zigzag curves, which are expensive to construct and maintain.

Details after Choosing Route

Final Location. With the route finally determined upon, it must be located. This consists in tracing the line, placing a stake at every 100 feet on the straight portions and at every 50 or 25 feet on the curves. At the tangent point of curves, and at points of compound and reverse curves a larger and more permanent stake should be placed. Lest those stakes should be disturbed in the process of construction, their exact distance from several points outside of the ground to be occupied by the road should be carefully measured and recorded in the notebook, so that they may be replaced. The stakes above referred to show the position of the center line of the road, and form the base line from which all operations of construction are carried on. Levels are taken at each stake, and cross levels are taken at every change of longitudinal slope.

Construction Profile. The construction or working profile is made from the levels obtained on location. It should be drawn to a horizontal scale of 400 feet to the inch and a vertical scale of 20 feet to the inch. Fig. 15 represents a portion of such a profile. The figures in column *A* represent the elevation of the ground at every 100 feet, or where a stake has been driven, above datum. The figures in column *B* are the elevations of the grade above datum. The figures in column *C* indicate the depth of cut or height of fill; they are obtained by taking the difference between the level of the road and the level of the surface of the ground. The straight line at the top represents the grade of the road; the upper surface of the road when finished would be somewhat higher than this, while the given line represents what is termed the *sub-grade or formation level*. All the dimensions refer to the formation level, to which the surface of the ground is to be formed to receive the road covering.

At all changes in the rate of inclination of the grade line a heavier vertical line should be drawn.

Gradient. *The grade of a line is in its longitudinal slope,* and is designated by the proportion between its length and the difference of height of its two extremes. The ratio of these two qualities gives it its name; if the road ascends or falls one foot in every twenty feet of its length, it is said to have a grade of 1 : 20, or a 5 per cent grade. Grades are of two kinds: maximum and minimum. The maximum grade is the steepest which is to be permitted and which on no account is to be exceeded; the minimum grade is the least allowable for good drainage. Table IX gives different methods of designating grades.

Determination of Gradients. The maximum grade is fixed by two considerations: the one relating to the power expended in

TABLE IX

Methods of Designating Grades

AMERICAN METHOD (ft. per 100 ft)	ENGLISH METHOD	FEET PER MILE	ANGLE WITH H
¼	1 : 400	13.2	0° 8′ 3
½	1 : 200	26.4	0 17 11
¾	1 : 150	39.6	0 22 5
1	1 : 100	52.8	0 34 2
1¼	1 : 80	66	0 42 5
1½	1 : 66⅔	79 2	0 51 2
1¾	1 : 57¼	92.4	1 0 51
2	1 : 50	105.6	1 8 6
2¼	1 : 44¼	118.8	1 17 3
2½	1 : 40	132	1 25 57
2¾	1 : 36⅛	145.2	1 34 22
3	1 : 33⅓	158.4	1 43 0
3¼	1 : 30¾	171.6	1 51 42
3½	1 : 28½	184 8	2 0 16
3¾	1 : 26⅔	198	2 8 51
4	1 : 25	211.2	1 17 26
4¼	1 : 23½	224.4	2 26 10
4½	1 : 22¼	237 6	2 34 36
4¾	1 : 21	250.8	2 43 35
5	1 : 20	264	2 51 44
6	1 : 12⅔	316 8	3 26 12
7	1 : 14²⁄₇	369.6	4 0 15
8	1 : 12½	422 4	4 34 26
9	1 : 11⅑	475 2	5 8 31
10	1 : 10	528	5 42 37

incline. There is a certain inclination, depending upon the
of perfection given to the surface of the road, which cann
exceeded without a direct loss of tractive power. This incli
is that, on which, in descending at a uniform speed, the
slacken, or which causes the vehicles to press on the horse
limiting inclination within which this effect does not take pl
the *angle of repose*.

Angle of Repose. The angle of repose for any given road s
can be ascertained easily from the tractive force required u
level with the same character of surface. Thus if the force nee
on a level to overcome the resistance of the load is $\frac{1}{30}$ of its w
then the same fraction expresses the angle of repose for that su

On all inclines less steep than the angle of repose, a c
amount of tractive force is necessary in the descent as well
the ascent, and the mean of the two drawing forces, ascendin
descending, is equal to the force along the level of the road.
on such inclines, as much mechanical force is gained in the

as is lost in the ascent. From this it might be inferred that when a vehicle passes alternately each way along the road, no real loss is occasioned by the inclination of the road; which, however, is not the fact with animal power, for while the up and down slopes in the ascending journey will demand, respectively, a greater and a less number of horses than that required on a level road, no actual compensation for this fluctuation can be made in the descending journey. On inclines which are more steep than the angle of repose, the load presses on the horses during their descent, so as to impede their action, and their power is expended in checking the descent of the load; or if this effect be prevented by the use of any form of drag or brake, then the power expended on such a drag or brake corresponds to an equal quantity of mechanical power expended in the ascent, for which no equivalent is obtained in the descent.

Grade Problems. *Maximum Grade.* The maximum grade for a given road will depend: (1) upon the class of traffic that will use it, whether fast and light, slow and heavy, or mixed, consisting of both light and heavy; (2) upon the character of the pavement adopted; and (3) upon the question of cost of construction. Economy of motive power and low cost of construction are antagonistic to each other, and the engineer will have to weigh the two in the balance.

For fast and light traffic the grades should not exceed 2 per cent; for mixed traffic 3 per cent may be adopted; while for slow traffic combined with economy 5 per cent should not be exceeded. This grade is practicable but not convenient.

Minimum Grade. From the previous considerations it would appear that an absolutely level road was the one to be sought for, but this is not so; there is a minimum, or a least allowable grade, of which the road must not fall short, as well as a maximum one which it must not exceed. If the road were perfectly level in its longitudinal direction, its surface could not be kept free from water without giving it so great a rise in its middle as would expose vehicles to the danger of overturning. The minimum grade commonly used is 1 per cent.

Undulating Grades. From the fact that the power required to move a load at a given velocity on a level road is decreased on a descending grade to the same extent it is increased in ascending the same grade, it must not be inferred that the animal force expended in passing alternately each way over a rising and falling road will gain as much in descending the several inclines as it will lose in ascending them. Such is not the case. The animal force must be sufficient, either in power or number, to draw the load over the level portions and up the steepest inclines of the road, and in practice no reduction in the number of horses can be made to correspond with the decreased power required in descending the inclines.

The popular theory that a gently undulating road is less fatiguing to horses than one which is perfectly level is erroneous. The assertion that the alternations of ascent, descent, and levels, call into play different muscles, allowing some to rest while others are exerted, and thus relieving each in turn, is demonstrably false, and contradicted by the anatomical structure of the horse. Since this doctrine is a mere popular error, it should be rejected utterly, not only because it is false in itself, but still more because it encourages the building of undulating roads, and this increases the labor and cost of transportation upon them.

Level Stretches. On long ascents it is generally recommended that level or nearly level stretches be introduced at frequent intervals in order to rest the animals. These are objectionable when they cause loss of height, and animals will be more rested by halting and unharnessing for half an hour than by traveling over a level portion. The only case which justifies the introduction of levels into an ascending road is where such levels will advance the road towards its objective point; where this is the case there will be no loss of either length or height, and it will simply be exchanging a level road below for a level road above.

Establishing the Grade. When the profile of a proposed route has been made, a grade line is drawn upon it (usually in red) in such a manner as to follow its general slope, but to average its irregular elevations and depressions. If the ratio between the whole distance and the height of the line is less than the maximum grade intended to be used, this line will be satisfactory; but if it be found steeper, the cut or the length of the line will have to be increased The latter is generally preferable.

The apex or meeting point of all curves should be rounded off by a vertical curve, as shown in Fig. 16. thus slightly changing the grade at and near the point of intersection. A vertical curve rarely need extend more than 200 feet each way from that point.

Let *AB* and *BC* be two grades in profile intersecting at station *B*, and let *A* and *C* be the adjacent stations. It is required to join the grades by a vertical curve extending from *A* to *C*. Imagine a chord drawn from *A* to *C*. The elevation of the middle point of the chord will be a mean of the elevations of the grade at *A* and *C*, and one-half of the difference between this and the elevation of the grade at *B* will be the middle ordinate of the curve. Hence we have

$$M=\frac{1}{2}\left(\frac{\text{grade } A+\text{grade } C}{2}-\text{grade} B\right)$$

in which *M* equals the correction in elevation for the point *B*. The correction for any other point is proportional to the square of its

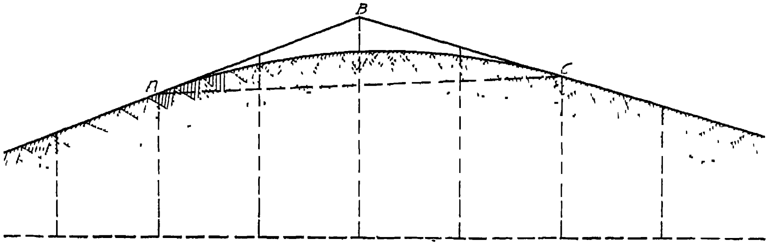


Fig. 16 Typical Road Section Showing Rounding Off of Meeting Points of Curves

distance from *A* to *C*. Thus, assuming the distance between successive ordinates, Fig. 16, as 50 feet, the correction *A*+25 is $\frac{1}{16}M$; at *A*+50 it is $\frac{1}{4}M$; at *A*+75 it is $\frac{9}{16}M$; and the same for corresponding points on the other side of *B*. The corrections in this case shown are subtractive, since *M* is negative. They are additional when *M* is positive, and the curve concave upward.

PRELIMINARY ROAD CONSTRUCTION METHODS

WIDTH AND TRANSVERSE CONTOUR

Width of Road. A road should be wide enough to accommodate the traffic for which it is intended, and should comprise a wheelway for vehicles and a space on each side for pedestrians.

The wheelway of country highways need be no wider than is places a track wide enough for a single team is all that is necessary. But the breadth of the land appropriated for highway purposes should be sufficient to provide for all future increase of traffic. The wheelways of roads in rural sections should be double; that is, one portion paved (preferably the center), and the other left with the natural soil. The latter, if kept in repair, will be preferred by teamsters for at least one-half the year.

The minimum width of the paved portion, if intended to carry two lines of travel, is fixed by the width required to allow two vehicles to pass each other safely. This width is 16 feet. If intended for a single line of travel, 8 feet is sufficient, but suitable turnouts must be provided at frequent intervals. The most economical width for any roadway is some multiple of eight. Wide roads are the best; they expose a larger surface to the drying action of the sun and wind, and require less supervision than narrow ones. Their first cost is greater than that of narrow ones, and nearly in the ratio of the increased width.

The cost of maintaining a mile of road depends more upon the extent of the traffic than upon the extent of its surface, and unless extremes be taken, the same quantity of material will be necessary for the repair of roads, either wide or narrow, which are subjected to the same amount of traffic. The cost of spreading materials over the wide road will be somewhat greater, but the cost of the materials will be the same. On narrow roads the traffic being confined to one track, will wear more severely than if spread over a wider surface.

The width of land appropriated for road purposes varies in the United States from 49½ feet to 66 feet; in England and France from 26 to 66 feet. And the width or space macadamized is also subject to variation; in the United States the average width is 16 feet; in France it varies between 16 and 22 feet; in Belgium 8¼ feet seems to be the regular width, while in Austria, from 11¼ to 26½ feet.

Transverse Contour. The centers of roadways in most cases should be higher than the sides, the object being to facilitate the flow of the rain water to the gutters. Where a good surface is maintained a very moderate amount of rise is sufficient for this purpose, but the rise should bear a certain proportion to the width

TABLE X

Proportionate Rise of Center to Width of Carriageway for Different Road Materials

KIND OF SURFACE	PROPORTIONS OF RISE AT CENTER TO WIDTH OF CARRIAGEWAY
Earth	1:40
Gravel	1:50
Broken stone	1:60

of the carriageway. Earth roads require the most and asphalt the least. The most suitable proportions for the different paving materials is shown in Table X.

Form of Contour. All authorities agree that the form should be convex, but they differ in the amount and form of the convexity. Circular arcs, two straight lines joined by a circular arc, and ellipses, all have their advocates. For country roads a curve of suitable convexity may be obtained as follows: At $\frac{1}{4}$ of the width from center to side, make the rise $\frac{7}{8}$ of the total rise, and at $\frac{1}{2}$ of the width make the rise $\frac{5}{8}$ of the total, Fig. 17.

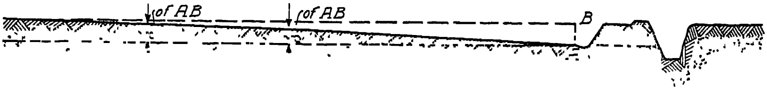


Fig. 17. Typical Section of Road, Showing Contour

Excessive height and convexity of cross section contract the width of the wheelway by concentrating the traffic at the center, that being the only part where a vehicle can run upright. The force required to haul vehicles over such cross sections is increased because an undue proportion of the load is thrown upon two wheels instead of being distributed equally over the four. The continual tread of horses' feet in one track soon forms a depression which holds water, and the surface is not so dry as with a flat section which allows the traffic to distribute itself over the whole width. Sides formed of straight lines are also objectionable. They wear hollow, retain water, and, by raising the center, defeat the object sought. The required convexity should be obtained by rounding the formation surface, and not by diminishing the thickness of the covering

Although on hillside and mountain roads it is generally recommended that the surface should consist of a single slope inclining inwards, there is no reason for or advantage gained by this method. The form best adapted to these roads is the same as for a road under ordinary conditions.

With a roadway raised in the center and the rain water draining off to gutters on each side, the drainage will be more effectual and speedy than if the drainage of the outer half of the road has to pass over the inner half. The inner half of such a road is usually subjected to more traffic than the outer half. If formed of a straight incline, this side will be worn hollow and retain water. The inclined flat section never can be properly repaired to withstand the traffic. Consequently it never can be kept in good order, no matter how constantly it may be mended. It is always below par and when heavy rain falls it is seriously damaged.

DRAINAGE

Types of Drainage. In the construction of roads, drainage is of the first importance. The ability of earth to sustain a load depends in a large measure upon the amount of moisture retained by it. Most earths form a good firm foundation so long as they are kept dry, but when wet they lose their sustaining power, becoming soft and incoherent.

The drainage of roadways is of two kinds, viz, subsurface and surface. The first provides for the removal of the underground water found in the body of the road; the second provides for the speedy removal of all water falling on the surface of the road. Experience has shown that a thorough removal of the underground water is of the utmost importance and is essential to the life of the road. A road covering placed on a wet undrained bottom will be destroyed by both water and frost, and will always be troublesome and expensive to maintain; perfect subsoil drainage is a necessity and will be found economical in the end even if in securing it considerable expense is required.

Subsoil Drainage

The methods employed for securing the subsoil drainage must be varied according to the character of natural soil, each kind of soil requiring different treatment.

Nature of Soils. The natural soil may be divided into the following classes: siliceous, argillaceous, and calcareous; rock, swamps, and morasses. The siliceous and calcareous soils, the sandy loams and rock, present no great difficulty in securing a dry and solid foundation. Ordinarily they are not retentive of water and therefore require no underdrains; ditches on each side of the road will generally be found sufficient. The argillaceous soils and softer marls require more care; they retain water and are difficult to compact, except at the surface; and they are very unstable under the action of water and frost.

Location of Drains. The removal of water from the subsoil is effected by drains so placed as to intercept the underground circulation of the water. Regarding the best location for the drains to accomplish this, three cases in general will present themselves:

Marginal Drains. Where the subsoil is continually wet and without a well-defined flow of water from either side. Under this condition marginal drains, as shown in Fig. 18, will be found satisfactory.

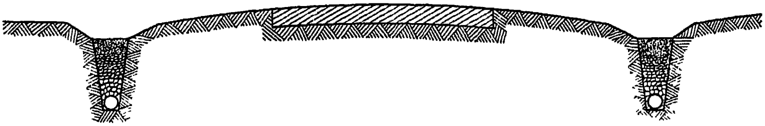


Fig. 18. Typical Road Section Showing Marginal Drains

Side Drains. Where there is a regular flow from one side to the other, as on a hillside road, a single drain placed on the side from which the water comes, as in Fig. 19, will be sufficient usually.

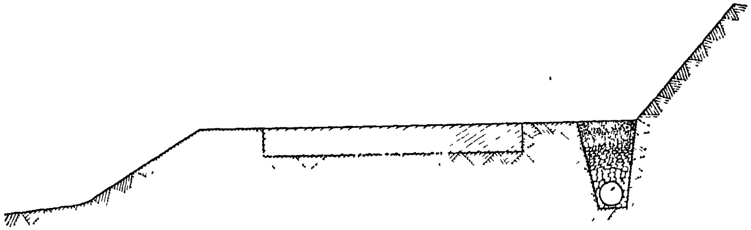


Fig. 19. Typical Road Section Showing Side Drains

Center and Cross Drains. Where the subsoil is so retentive of water as to require a system of drains under the roadbed, these drains may be constructed in a variety of ways. The simplest method is to place a drain under the center of the roadway, as in Fig. 20, connecting it at intervals by cross drains with drains placed at the sides which discharge into the natural watercourses. Where the ground is level or has but a slight inclination, the cross drains may be placed at right angles to the axis of the road. Where there is a steep grade it is better to lay the cross drains in the form of an inverted V with the point in the center of the roadway and directed uphill.

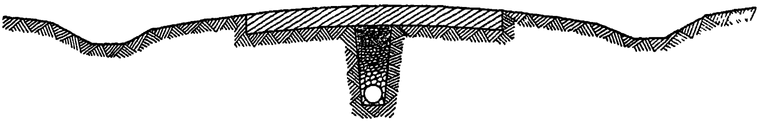


Fig 20. Typical Road Section Showing Center Drain

The distance apart of the cross drains depends upon the ease with which the subsoil yields its water. In porous soils the drains will prove efficient at distances of from 30 to 40 feet; in retentive clay the spacing may range from 10 to 20 feet.

Proper Fall for Drains. The fall to be given the drains depends upon the size of the drain and the amount of water to be carried off. It is not advisable to employ a fall greater than 1 foot in 100 feet. Too great a fall will produce a swift current that is liable to undermine the drain as well as to choke it by foreign matter, which a less rapid stream could not have transported.

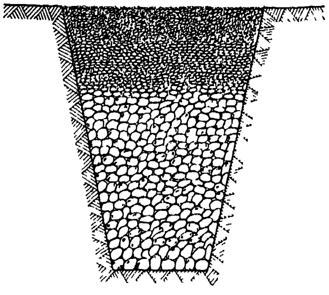


Fig 21. "Blind" Stone Drain

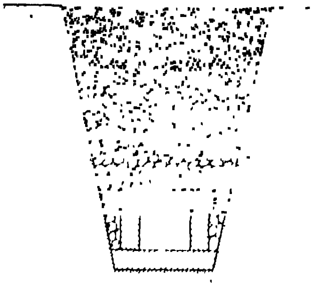


Fig 22 "Throat" Stone Drain

Materials Used for Drains. The materials employed for drains are: stone, vitrified clay pipe, porous tile, and concrete.

Stone Drains. The stone drain is constructed in two forms, shown in Figs. 21 and 22. The first form, called a "blind" drain, consists of a trench excavated to the required depth and filled with cobblestones or rounded pebbles. To prevent the soil from washing in and choking it, the larger stones are covered first with a layer of small gravel, and then with a layer of coarse gravel, by which means the water is filtered before passing into the porous mass beneath. Angular stones are not suitable for this type of drain. The second form of stone drain, an open channel called a "throat", is formed in the bottom of the trench with rough slabs of stone, and the trench is filled in the same manner as for a blind drain.

Vitrified Pipe Drains. Vitrified pipe drains are constructed by placing the pipe in the bottom of the trench, filling the hubs with oakum and back-filling the trench with gravel, broken stone, or a mixture of these.

Porous Tile Drains. Porous tile, Fig. 23, form very satisfactory drains. They carry off the water with great ease, rarely if ever get choked, and require only a slight inclination to keep the water moving through them. The tile have plain ends which are placed in contact in the trench and wrapped with tar paper or burlap. They are surrounded and covered with gravel or broken stone not exceeding 1 inch in size for a depth ranging from 6 to 12 inches, and the remaining depth of the trench is filled with large gravel or broken stone.

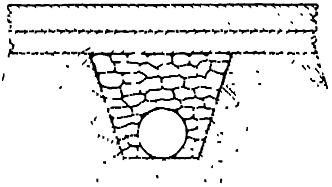


Fig 23 Porous Tile Drain

Concrete Tile Drains. Tile made of concrete have been in satisfactory service for several decades. They are generally made in lengths of one foot with plain ends and are laid in the same manner as the porous tile. They can be made in portable machines in the vicinity where they are to be used—an advantage that tends toward low first cost.

For the manufacture of concrete tile the best quality of hydraulic (Portland) cement, clean sand, and fine gravel or broken stone should be used in the proportion of one part cement, two ticles exceeding $\frac{3}{4}$ inch in size. Sufficient clean water should be used to produce a "wet mix", which should be poured into the molds before setting begins and rammed lightly. After setting is completed, the tile should be cured for about 90 days.

Sizes of Drains. The size of the drain to be adopted for a given situation depends upon the amount of water to be carried and the fall that can be given the drain. These two factors being given, there are several formulas that can be used to determine the required size. But, in the subsoil drainage of a road, the amount of water to be moved can be guessed at only; therefore, experience as to what a drain has accomplished in a given locality is a better guide than the result given by any formula. Experience shows that the least practicable size is 4 inches. The amount of water to be moved is generally assumed to vary between $\frac{1}{4}$ inch and 1 inch per acre, per 24 hours, on the area to be drained.

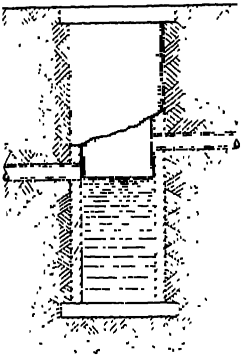


Fig. 24. Typical Construction for Silt Basin

Silt Basins. Silt basins should be constructed at all junctions and wherever else they may be considered necessary; they may be made from a single 6-inch pipe, Fig. 24, or constructed of brick masonry.

Protection of Drain Ends from Weather. As tile drains are more liable to injury from frost than those of either brick or stone, their ends at the side ditches should not be exposed directly to the weather in very cold climates, but may terminate in blind

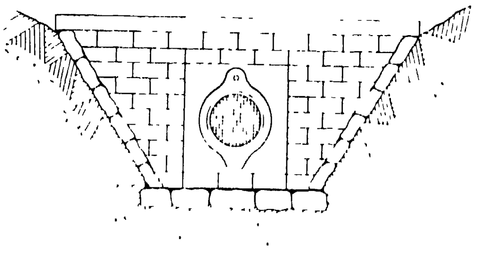


Fig 25 Proper Method of Covering Drain Outlet

drains, or a few lengths of vitrified clay pipe reaching under the road a distance of about 3 or 4 feet from the inner slope of the ditch.

Drain Outlets. Drain outlets may be formed by building a dwarf wall of brick or stone, whichever is the cheapest or most convenient in the locality. The outlet, Fig. 25, should be covered with an iron grating to prevent vermin entering the drain pipes and building nests, thus choking the waterway.

Side Ditches. Side ditches are provided to carry away the subsoil water from the base of the road, and the rain water which falls upon its surface; to do this speedily they must have capacity and inclination proportionate to the amount of water reaching them. The width of the bed should not be less than 18 inches; the depth will vary with circumstances, but should be such that the water surface shall not reach the subgrade, but remain at least 12 inches below the crown of the road. The sides should slope at least $1\frac{1}{2}$ to 1.

The longitudinal inclination of the ditch follows the configuration of the general topography, that is, the lines of natural drainage. When the latter has to be aided artificially, grades of from 1 in 500 to 1 in 800 will usually answer.

In absorbent soil less fall is sufficient, and in certain cases level ditches are permissible. The slopes of the ditches must be protected where the grade is considerable. This can be accomplished by sod revetments, riprapping, or paving.

Surface Drainage

The drainage of the roadway surface depends upon the preservation of the cross section, with regular and uninterrupted fall to the sides and without hollows or ruts in which the water can lie, and also upon the longitudinal fall of the road. If this is not sufficient the road becomes flooded during heavy rainstorms and melting snow, and is considerably damaged.

Side Ditches and Gutters. The removal of surface water from country roads may be effected by the side ditches, into which, when there are no sidewalks, the water flows directly. When there are sidewalks, gutters are formed between the roadway and footpath, and the water is conducted from these gutters into the side ditches by tile piping laid under the walks at intervals of about

50 feet. The entrance to these pipes should be protected against washing by a rough stone paving. In the case of covered ditches under the footpath, the water must be led into them by first passing through catch basins. These are small masonry vaults covered with iron gratings to prevent the ingress of stones, leaves, etc. Connection from the catch basin is made by a tile pipe about 6 inches in diameter. The mouth of this pipe is placed a few feet above the bottom of the catch basin, and the space below it acts as a depository for the silt carried by the water, and is cleaned out periodically. The catch basins may be placed from 200 to 300 feet apart. They should be made of dimensions sufficient to convey the amount of water which is liable to flow into them during heavy and continuous rains.

If on inclines the velocity of the water is greater than the nature of the soil will withstand, the gutters should be roughly paved. cases, the slope adjoining the footpath should be covered sod. A velocity of 30 feet a minute will not disturb clay with sand and stone; 40 feet per minute will move coarse sand; 60 feet a minute will move gravel; 120 feet a minute should move round pebbles 1 inch in diameter, and 180 feet a minute will move angular stones $1\frac{3}{4}$ inches in diameter.

The scour in the gutters on inclines may be prevented by small weirs of stones or wood stick fascines constructed by the roadmen at a nominal cost. At junctions and crossroads the gutters and side ditches require careful arrangement so that the water from one road may not be thrown upon another; cross drains and culverts will be required at such places.

Treatment of Springs Found in Cuttings. In cuttings, springs are frequently encountered and become a source of constant danger to the stability of the slopes. In such cases the slope should be excavated at the point where the water appears, until, if possible, the source is reached. When the source has been reached, an outlet is provided by constructing a drain and connecting it with the drain at the roadside. Sometimes it may be impossible to trace the water to a single source, the whole face of the cutting being saturated for some distance. In such cases the treatment may be difficult and expensive, but a series of drains may be run up the slope to such height as will tap all the water appearing.

In cuttings, the ditch at the toe of the slope is liable to be filled with silt carried down the slope by rain; and where this might occur, covered drains should be constructed.

Drainage for Hillside Roads. On hillside or mountain roads catch-water ditches should be cut on the mountain side above the road, to cut off and convey the drainage of the ground above them to the neighboring ravines. The size of these ditches will be determined by the amount of rainfall, extent of drainage from the mountain which they intercept, and by the distances of the ravine watercourses on each side.

Inner and Outer Road Gutters. The inner road gutter should be of dimensions ample to carry off the water reaching it; when in soil, it should be roughly paved with stone. When paving is not absolutely necessary, but is desirable to arrest the scouring action of running water during heavy rains, stone weirs may be erected across the gutter at convenient intervals. The outer gutter need not be more than 12 inches wide and 9 inches deep. The gutter is formed by a depression in the surface of the road close to the parapet or revetted earthen protection mound. The drainage which falls into this gutter is led off through the parapet, or other roadside protection, at frequent intervals. The guard stones on the outside of the road are placed in and across the gutter, just below the drainage holes, so as to turn the current of the drainage into these holes or channels. On straight reaches, with parapet protection, drainage holes with guard stones should be placed every 20 feet apart. Where earthen mounds are used, and it may not be convenient to have the drainage holes or channels every 20 feet, the guardstones are to be placed in advance of the gutter to allow the drainage to pass behind them. This drainage is either to be run off at the cross drainage of the road, or to be turned off as before by a guard stone set across the gutter.

At re-entering turns, where the outer side of the road requires particular protection, guard stones should be placed every 4 feet. As all re-entering turns should be protected by parapets, the drainage holes through them may be placed as close together as desired.

Where the road is in embankment the surface water must be prevented from running down the slopes by providing ample gutters

Water Breaks. Water breaks to turn the surface drainage into the side ditches, should not be constructed on improved roads. They increase the grade and are an impediment to convenient and easy travel. Where it is necessary that water should cross the road, a culvert should be built.

CULVERTS

Functions of Culverts. Culverts are necessary for carrying the cross streams under a road, and also for conveying the surface water collected in the side ditches from the upper side to that side on which the natural watercourses lie.

Especial care is required to provide an ample way for the water to be passed. If the culvert is too small, it is liable to cause a washout, entailing interruption of traffic and cost of repairs, and possibly may cause accidents that will require payment of large sums for damages. On the other hand, if the culvert is made ssarily large, the cost of construction is needlessly increased.

Factors Considered in Design of Culverts. The area of waterway required depends upon a number of important factors, which will be discussed briefly.

Rate of Rainfall. It is the maximum rate of rainfall during the severest storms which is required in this connection. This varies greatly in different sections of the country.

The maximum rainfall as shown by statistics is about one inch per hour (except during heavy storms); equal to 3,630 cubic feet per acre. Owing to various causes, not more than 50 to 75 per cent of this amount will reach the culvert within the same hour.

Inches of rainfall × 3,630 = cubic feet per acre
Inches of rainfall × 2,323,200 = cubic feet per square mile

Kind and Condition of Soil. The amount of water to be drained off will depend upon the permeability of the surface of the ground, which will vary greatly with the kind of soil, the degree of saturation, the condition of the cultivation, the amount of vegetation, etc.

Character and Inclination of Surface. The rapidity with which the water will reach the watercourse depends upon whether the surface is rough or smooth, steep or flat, barren or covered with vegetation, etc.

Condition and Inclination of Stream Bed. The rapidity with which the water will reach the culvert depends upon whether there is a well-defined and unobstructed channel or whether the water finds its way in a broad, thin sheet. If the watercourse is unobstructed and has a considerable inclination, the water may arrive at the culvert nearly as rapidly as it falls; but if the channel is obstructed, the water may be much longer in passing the culvert than in falling.

Shape of Area to be Drained and Position of Stream Branches. The area of waterway depends upon the amount of the area to be drained; but in many cases the shape of this area and the position of the branches of the stream are of more importance than the amount of the territory. For example, if the area is long and narrow, the water from the lower portion may pass through the culvert before that from the upper end arrives; or, on the other hand, if the upper end of the area is steeper than the lower, the water from the former may arrive simultaneously with that from the latter. Again, if the lower part of the area is supplied better with branches than the upper portion, the water from the former will be carried past the culvert before the arrival of that from the latter; or, on the other hand, if the upper part is supplied better with branch watercourses than is the lower, the water from the whole area may arrive at the culvert at nearly the same time. In large areas the shape of the area and the position of the watercourses are very important considerations.

Mouth of Culvert and Inclination of Bed. The efficiency of a culvert may be increased very materially by arranging the upper end so that the water may enter into it without being retarded. The discharging capacity of a culvert can be increased greatly by increasing the inclination of its bed, provided the channel below will allow the water to flow away freely after having passed the culvert.

Provision for Discharge of Water Under Head. The discharging capacity of a culvert can be increased greatly by allowing the water to dam up above it. A culvert will discharge twice as much under a head of four feet as under a head of one foot. This can be done safely only with a well-constructed culvert.

The determination of the values of the different factors entering into the problem is almost wholly a matter of judgment. An estimate for any one of the above factors is liable to be in error from 100 to 200 per cent, or even more, and of course any result mathematical exactness is not required by the problem nor warranted by the data. The question is not one of 10 or 20 per cent of increase; for if a 2-foot pipe is insufficient, a 3-foot pipe probably will be the next size, an increase of 225 per cent; and if a 6-foot arch culvert is too small, an 8-foot will be used, an increase of 180 per cent. The real question is whether a 2-foot pipe or an 8-foot arch culvert is needed.

Valuable data on the proper size of any particular culvert may be obtained as follows: (1) by observing the existing openings on the same stream; (2) by measuring, preferably at time of high water, a cross section of the stream at some narrow place; and (3) by determining the height of high water as indicated by drift and débris, and from the evidence of the inhabitants of the neighborhood.

On mountain roads, or roads subjected to heavy rainfall, culverts of ample dimensions should be provided wherever required, and it will be more economical to construct them of masonry. In localities where boulders and débris are likely to be washed down during wet weather, it will be a good precaution to construct catch pools at the entrance of all culverts and cross drains for the reception of such matter. In hard soil or rock these catch pools will be simple well-like excavations, with their bottoms two or three feet below the entrance sill or floor of the culvert or drain. Where the soil is soft they should be lined with stone laid dry; if very soft, with masonry. The size of the catch pools will depend upon the width of the drainage works. They should be wide enough to prevent the drains from being injured by falling rocks and stones of a not inordinate size.

The use of catch pools obviates the necessity of building culverts and drains at an angle to the axis of the road. Oblique structures are objectionable, as being longer than if set at right angles and by reason of the acute- and obtuse-angled terminations to their piers, abutments, and coverings.

Types of Culverts

General Classification. Three types of culverts are employed, namely: pipe, box, and arch. The pipe culvert is employed for small streams, in sizes from 12 to 24 inches. Box culverts are employed in sizes from 24 inches up to 8 feet. Arch culverts are used for spans 8 feet and over. In the construction of culverts a variety of materials are used. Pipe culverts are constructed of earthenware or vitrified clay, cast iron, corrugated steel, brick, or concrete; box and arch culverts are built of stone, brick, or concrete. Short span concrete bridges are also often employed as culverts. The type of culvert and the material to be used are determined in some cases by the cost; in others by the load to be supported, as where the depth of fill over the culvert is considerable, or where a large area of waterway is required.

Earthenware Pipe Culverts. *Construction.* In laying the pipe the bottom of the trench should be rounded out to fit the lower half of the body of the pipe, with proper depressions for the sockets. If the ground is soft or sandy, the earth should be rammed carefully, but solidly, in and around the lower part of the pipe. The top surface of the pipe, as a rule, never should be less than 18 inches below the surface of the roadway, but there are many cases where pipes have stood for several years, under heavy loads, with only 8 to 12 inches of earth over them. No danger from frost need be apprehended, provided the culverts are so constructed that the water is carried away from the level end. Ordinary soft drain tiles are not affected in the least by the expansion of frost in the earth around them.

The freezing of water in the pipe, particularly if more than half full, is liable to burst it; consequently the pipe should have a sufficient fall to drain itself, and the outside should be so low that there is no danger of backwaters reaching the pipe. If properly drained, there is no danger from frost.

Jointing. In many cases, perhaps in most, the joints are not calked. If this is not done, there is danger of the water being forced out of the joints and of washing away the soil from around the pipe. Even if the danger is not very imminent, the joints of the larger pipes, at least, should be calked with hydraulic cement, since the cost is very small compared with the insurance against damage thereby secured. Sometimes the joints are calked with clay. Every culvert should be built so it can discharge water under a head without damage to itself.

Use of Bulkheads. Although often omitted, the end sections

The foundation of the bulkhead should be deep enough not to be disturbed by frost. In constructing the end wall, it is well to increase the fall near the outlet to allow for a possible settlement

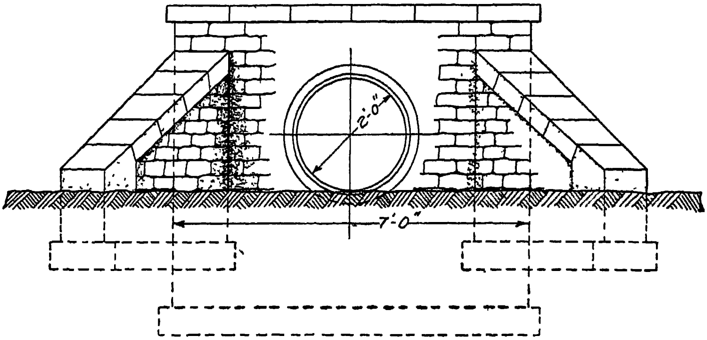


Fig. 26. Typical Design for Masonry Bulkhead

of the interior sections. When stone and brick abutments are too expensive, a fair substitute can be made by setting posts in the ground and spiking plank to them. When planks are used, it is best to set them with considerable inclination towards the roadbed to prevent their being crowded outward by the pressure of the embankment. The upper end of the culvert should be so protected

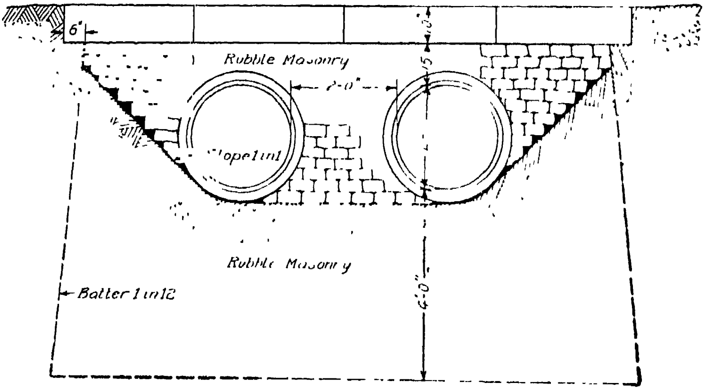


Fig 27. Section Showing Typical Layout for Double Pipe Culvert

that the water will not readily find its way along the outside of the pipes, in case the mouth of the culvert should become submerged.

When the capacity of one pipe is not sufficient, two or more may be laid side by side as shown in Fig. 27. Although the two small pipes do not have as much discharging capacity as a single large one of equal cross section, yet there is an advantage in laying two small ones side by side, since the water need not rise so high to utilize the full capacity of the two pipes as would be necessary to discharge itself through a single one of large size.

Iron Pipe Culverts. During recent years iron pipe, Fig. 28, has been used for culverts on many prominent railroads, and may be used on roads in sections where other materials are unavailable.

In constructing a culvert with cast-iron pipe the points requiring particular attention are: (1) tamping the soil tightly around the pipe to prevent the water from forming a channel along the outside; and (2) protecting the ends by suitable head walls and, when necessary, laying riprap at the lower end. The amount of masonry required for the end walls depends upon the relative width of the embankment and the number of sections of pipe used. For example, if the embankment is, say, 40 feet wide at the base, the culvert may consist of three 12-foot lengths of pipe and a light end wall near the toe of the bank; but if the embankment is, say, 32 feet wide, the culvert may consist of two 12-foot lengths of pipe and a comparatively heavy end wall well back from the toe of the bank. The smaller sizes of pipe usually come in 12-foot lengths, but sometimes a few 6-foot lengths are included for use in adjusting the length of the culvert to the width of the bank. The larger sizes

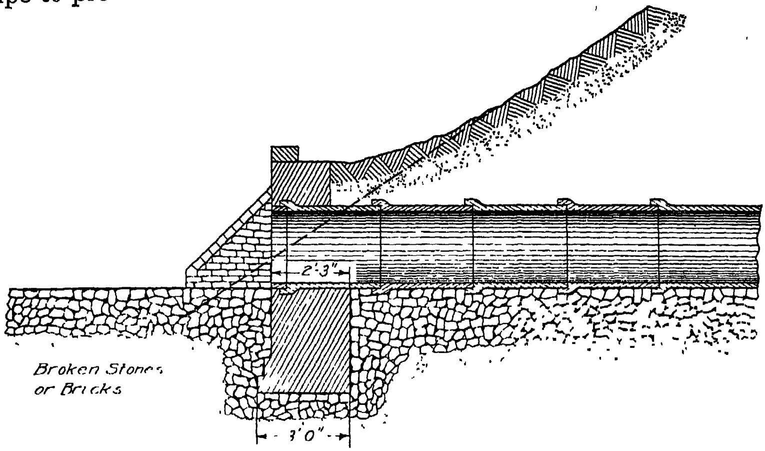


Fig. 28. Section Showing Construction of Iron Pipe Culvert

Box Culverts. Box culverts, Fig. 29, consist of two side walls with a flat deck. When stone is used, they are generally built of dry rubble masonry. The walls should be well founded at about 42 inches below the bed of the stream. The thickness of the walls varies according to the height. The wings are formed by extending the walls out straight and stepping them down. The deck may be made of stone slabs or reinforced concrete; with the latter it is possible to use wider spans than with stone slabs. Where the force of the stream is sufficient to scour the bed, it will be necessary to pave it with stone or concrete. When reinforced concrete is used instead of stone, the side walls are made from 4 to 8 inches thick, depending upon the height. Where it is not necessary to pave the stream bed, the walls are carried down about 2 feet below the bed, and founded upon a footing 9 to 12 inches thick and sufficiently wide to secure ample area of the soil to support the load. Where scouring of the bed is liable to occur, a concrete bottom is constructed throughout the entire width and length of the culvert, and the side walls are founded on it: if necessary, a cut-off wall

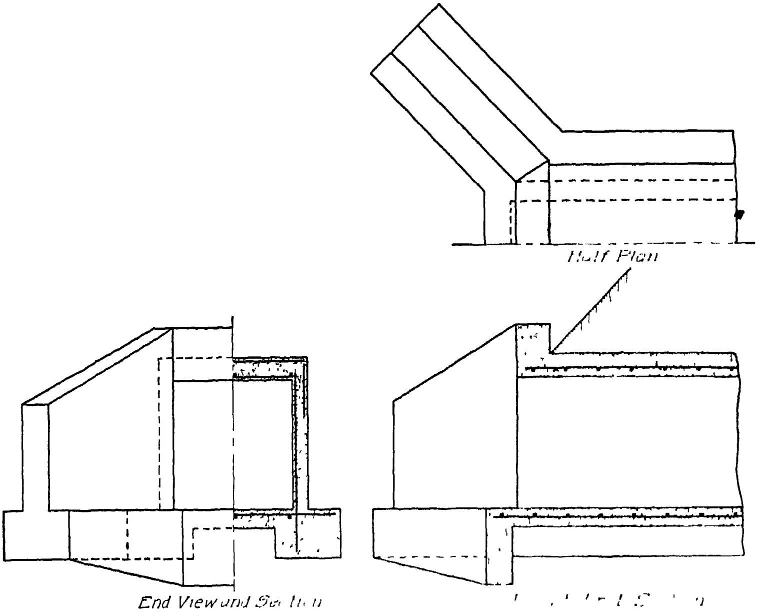


Fig. 29 Typical Design of Concrete Box Culvert

is constructed across each end to a depth of about 2 feet below the bottom.

Arch Culverts. The arch form of culvert is more costly than the other forms, but it is often preferred on account of its appearance, Fig. 30. When masonry and plain concrete are used, very heavy abutments are required in order that no movement can take place under a live load, to cause bending moments in the arch. In designing reinforced-concrete arches, bending is provided for by considering the arch as a curved beam, with a consequent reduction in the weight of the abutments.

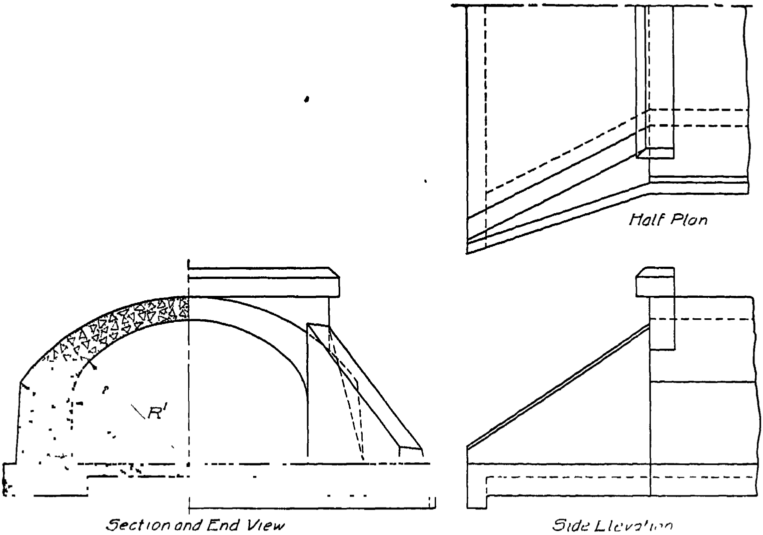
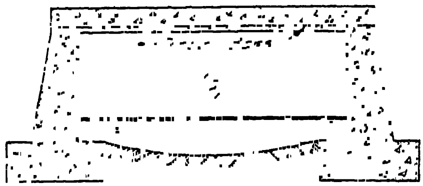


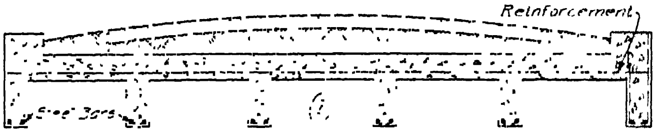
Fig. 30. Design for Arch Culvert

Short Span Bridges Used as Culverts. Three types of reinforced-concrete bridges are employed for short spans: (*a*) the flat slab; (*b*) the T-beam; (*c*) the steel I-beam incased in concrete, Fig. 31. The length of span over which reinforced slabs may be built with safety depends upon the load to be carried; under normal conditions the maximum span is 12 feet. The thickness of the slab for a span 2 feet should be not less than 6 inches and should increase with increase of span. The slabs are reinforced with steel bars, expanded metal, or other forms of reinforcing metal; the cross-sectional area of the reinforcing steel required is about 1 per cent of that of the slab.*

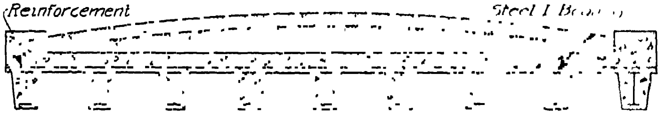
The T-beam type is practicable for spans from 12 to 30 feet. The I-beam type may be used for all spans up to 30 feet. In this type, the I-beam is designed to transmit the load to the abutments, while the reinforced-concrete floor transmits the load to the I-beams. This type of construction is noted for its safety and ability to withstand severe and unfavorable conditions, such as the settlement of the abutments, which may cause rupture of the concrete. The I-beam may or may not be incased in the concrete; the object sought in so doing is to protect it from rust. This may be accomplished also by painting, but as this needs to be repeated frequently and as there is a possibility that it will not be done, it is better to incase the beams in the concrete during construction, and so insure their permanent protection. This type also admits of arch construction between the beams for the floor system, thus decreasing the depth required for the floor; this feature may be of value in locations where the area of the waterway or the "head room" is a controlling factor.



Section of Flat Slab Culvert.



Section Through Top For T Beam Culvert



Section Through Top For I Beam Culvert

Fig. 31 Sections of Typical Short Span Concrete Bridges Used as Culverts

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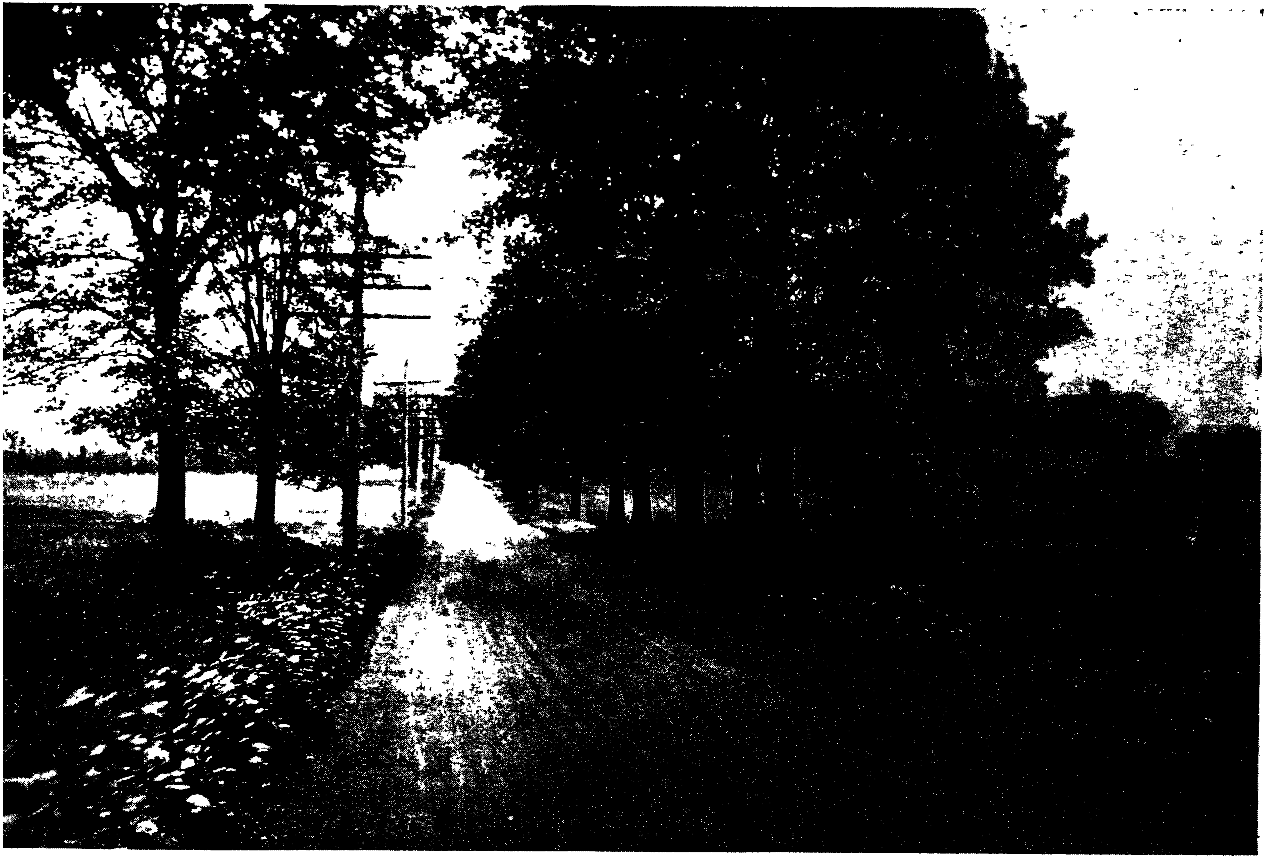
*The theory of design of concrete bridges and culverts is discussed in Masonry and Reinforced Concrete, Part III



VIEW OF NEW YORK STATE HIGHWAY FROM ALBANY TO SELKIRK--TYPICAL TARVIA DUSTLESS ROAD

Courtesy of Barrett Manufacturing Company, New York City, New York

EARTHWORK

The term "earthwork" is applied to all the operations performed in the making of excavations and embankments. In its widest sense it comprehends work in rock as well as in the looser materials of the earth's crust.

Balancing Cuts and Fills. In the construction of new roads, the formation of the roadbed consists in bringing the surface of the ground to the adopted grade. This grade should be established so as to reduce the earthwork to the least possible amount, both to render the cost of construction low, and to avoid unnecessarily marring the appearance of the country in the vicinity of the road. The most desirable position of the grade line is usually that which makes the amounts of cutting and filling equal to each other, for any surplus embankment over cutting must be made up by borrowing, and surplus cutting must be wasted; both of these operations involving additional cost for labor and land.

Side Slopes. *Inclination.* The proper inclination for the side slopes of cutting and embankments depends upon the nature of the soil, the action of the atmosphere, and the action of internal moisture upon it. For economy the inclination should be as steep as the nature of the soil will permit.

The usual slopes in cuttings are:

Solid rock	$\frac{1}{4}$ to 1
Earth and gravel	$1\frac{1}{2}$ to 1
Clay	3 or 6 to 1
Fine sand	2 or 3 to 1

The slopes of embankment are usually made $1\frac{1}{2}$ to 1.

Form of Slopes. The natural, strongest, and ultimate form of earth slopes is a concave curve, in which the flattest portion is at the bottom. This form is very rarely given to the slopes in constructing them; in fact, the reverse is often the case, the slopes being made convex, thus saving excavation by the contractor and inviting slips.

In cuttings exceeding 10 feet in depth the forming of concave slopes will aid materially in preventing slips, and in any case they will reduce the amount of material which eventually will have to be removed when cleaning up. Straight or convex slopes will continue to slip until the natural form is attained

A revetment or retaining wall at the base of a slope will save excavation.

In excavations of considerable depth, and particularly in soils liable to slips, the slope may be formed in terraces, the horizontal offsets or benches being made a few feet in width with a ditch on the inner side to receive the surface water from the portion of the side slope above them. These benches catch and retain earth that may fall from the slopes above them. The correct forms for the slopes of embankment and excavation are shown in Figs. 32 and 33.

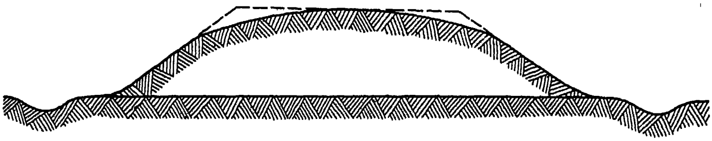


Fig. 32. Section Showing Correct Slopes of Embankments

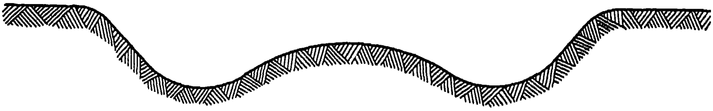


Fig 33. Section Showing Correct Slopes of Excavations

Covering of Slopes. It is not usual to employ any artificial means to protect the surface of the side slopes from the action of the weather; but it is a precaution which in the end will save much labor and expense in keeping the roadways in good order. The simplest means which can be used for this purpose consist in covering the slopes with good sods, or else with a layer of vegetable mold about four inches thick, carefully laid and sown with grass seed. These means are amply sufficient to protect the side slopes from injury when they are not exposed to any other cause of deterioration than the wash of the rain and the action of frost on the ordinary moisture retained by the soil.

A covering of brushwood or a thatch of straw may also be used with good effect; but from their perishable nature they will require frequent renewal and repairs.

Where stone is abundant a small wall of stone laid dry may be constructed at the foot of the slopes to prevent any wash from them being carried into the ditches.

Shrinkage of Earthwork. All materials when excavated increase in bulk, but after being deposited in banks subside or shrink (rock excepted) until they occupy less space than in the pit from which excavated.

Rock, on the other hand, increases in volume by being broken up, and does not settle again into less than its original bulk. The increase may be taken at 50 per cent.

The shrinkage in the different materials is about as follows:

Gravel	8 per cent
Gravel and sand	9 per cent
Clay and clay earths	10 per cent
Loam and light sandy earths	12 per cent
Loose vegetable soil	15 per cent
Puddled clay	25 per cent

Thus an excavation of loam measuring 1000 cubic yards will form only about 880 cubic yards of embankment, or an embankment of 1000 cubic yards will require about 1120 cubic yards, measured in excavation, to make it. A rock excavation measuring 1000 yards will make from 1500 to 1700 cubic yards of embankment, depending upon the size of the fragments.

The lineal settlement of earth embankments will be about in the ratio given above; therefore either the contractor should be instructed, in setting his poles to guide him as to the height of grade on an earth embankment, to add the required percentage to the fill marked on the stakes, or the percentage may be included in the fill marked on the stakes. In rock embankments this is not necessary.

Classification of Earthwork. Excavation is usually classified as earth, hardpan, loose rock, or solid rock. For each of these classes a specific price is usually agreed upon, and an extra allowance is sometimes made when the haul, or distance to which the excavated material is moved, exceeds a given amount.

The characteristics which determine the classes to which a given material belongs are usually described with clearness in the specifications, as:

Earth, to include loam, clay, sand, and loose gravel.

Hardpan, to include cemented gravel, slate, cobbles, and boulders containing less than 1 cubic foot, and all other material

Loose rock, to include shale, decomposed rock, boulders, and detached masses of rock containing not less than 3 cubic feet, and all other material of a rock nature which may be loosened with a pick, although blasting may be resorted to in order to expedite the work.

Solid rock, to include all rock found in place in ledges and masses, or boulders measuring more than 3 cubic feet, and which can only be removed by blasting.

Prosecution of Earthwork. No general rule can be laid down for the exact method of carrying on an excavation and disposing of the excavated material. The operation in each case can be determined only by the requirements of the contract, character of the material, magnitude of the work, length of haul, etc.

Methods of Forming Embankments. *General Case.* Where embankments are to be formed less than 2 feet in height, all stumps, weeds, etc., should be removed from the space to be occupied by the embankment. For embankments exceeding 2 feet in height stumps need only be close cut. Weeds and brush, however, ought to be removed and if the surface is covered with grass sod, it is advisable to plow a furrow at the toe of the slope. Where a cut passes into a fill all the vegetable matter should be removed from the surface before placing the fill. The site of the bank should be examined carefully and all deposits of soft, compressible matter removed. When a bank is to be made over a swamp or marsh, the site should be drained thoroughly, and if possible the fill should be started on hard bottom.

Perfect stability is the object aimed at, and all precautions necessary to this end should be taken. Embankments should be built in successive layers: banks 2 feet and under in layers from 6 inches to 1 foot; heavier banks in layers 2 and 3 feet thick. The horses and vehicles conveying the materials should be required to pass over the bank for the purpose of consolidating it, and care should be taken to have the layers dip towards the center. Embankments which have been first built up in the center, and afterwards widened by dumping the earth over the sides, should never be allowed.

Embankments on Hillsides. When the axis of the road is laid out on the side slope of a hill and the road is formed partly by excavating and partly by embanking, the usual and most simple method is to extend out the embankment gradually along the whole line of the excavation. This method is insecure; the excavated material if simply deposited on the natural slope is liable to slip, and no pains should be spared to give it a secure hold, particularly at the toe of the slope. The natural surface of the slope should be cut into steps, as shown in Fig. 34. The dotted line *AB* represents the natural surface of the ground, *CEB* the excavation, and *ADC* the embankment, resting on steps which have been cut between *A* and *C*. The best position for these steps is perpendicular to the axis of greatest pressure. If *AD* is inclined at the angle of repose of the material, the steps near *A* should be inclined in the opposite direction to *AD*, and at an angle of nearly 90 degrees thereto, while the steps near *C* may be level. If stone is abundant in the locality, the toe of the slope may be further secured by a dry wall of stone.

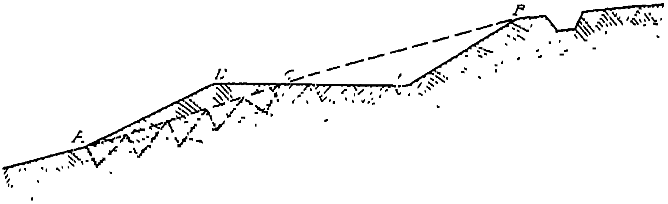


Fig. 34 Section of Embankment Showing Reinforcing by Means of Steps

On hillsides of great inclination the above method of construction will not be sufficiently secure; retaining walls of stone must be substituted for the side slopes of both the excavations and embankments. These walls may be made of stone laid dry, when stone can be procured in blocks of sufficient size to render this kind of construction of sufficient stability to resist the pressure of the earth. When the stones laid dry do not offer this security, they must be laid in mortar. The wall which forms the slope of the excavation should be carried up as high as the natural surface of the ground. Unless the material is such that the slope may be safely formed into steps or benches, as shown in Fig. 34, the wall of the roadway, and a parapet wall or fence raised upon it, to protect pedestrians against accident, Fig. 35.

For the formula for calculating the dimensions of retaining walls see Instruction Paper on Masonry and Reinforced Concrete, Part III.

Treatment of Roadways on Rock Slopes. On rock slopes, when the inclination of the natural surface is not greater than 1 on the vertical to 2 on the base, the road may be constructed partly in excavation and partly in embankment in the usual manner, or by cutting the face of the slope into horizontal steps with vertical faces, and building up the embankment in the form of a solid stone wall in horizontal courses, laid either dry or in mortar. Care is required in proportioning the steps, as all attempts to lessen the quantity of excavation, by increasing the number and diminishing the width of the steps, require additional precautions against settlement in the built-up portion of the roadway.

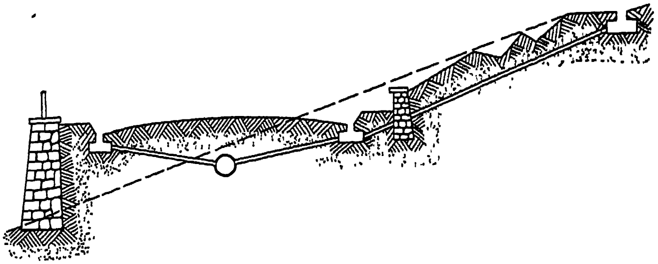


Fig 35. Reinforcing Roadway by Parapet Wall or Fence

When the rock slope has a greater inclination than 1:2 the whole of the roadway should be in excavation.

In some localities roads have been constructed along the face of nearly perpendicular cliffs, on timber frameworks consisting of horizontal beams firmly fixed at one end by being let into holes drilled in the rock, the other end being supported by an inclined strut resting against the rock in a shoulder cut to receive it. There are also examples of similar platforms suspended instead of being supported.

Tools for Construction Work

Picks. Picks are made in various styles, according to the class of material in which they are to be used. Fig. 36 shows the form usually employed in street work. Fig. 37 shows the form generally used for clay or gravel excavation.

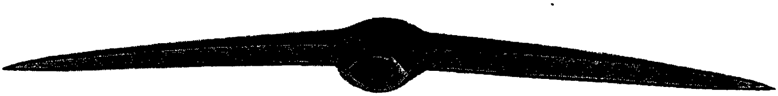


Fig. 36. Grading Pick
Courtesy of Acme Road Machinery Company, Frankfort, New York

The eye of the pick is formed generally of wrought iron, while the points are of steel. The weight of picks ranges from 4 to 9 pounds.

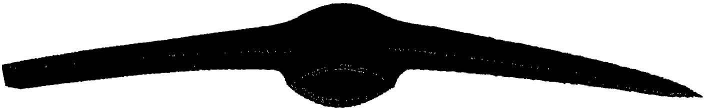


Fig. 37. Clay Pick
Courtesy of Acme Road Machinery Company, Frankfort, New York

Grubbing Tools. In handling brush, stumps, etc., such tools as the bush hooks, Fig. 38, the bush mattock, Fig. 39, and the axe mattock, Fig. 40, are commonly used. These are cutting as well as grading tools.

Shovels. Shovels, Fig. 41, are made in two forms, square and round pointed, usually of pressed steel.

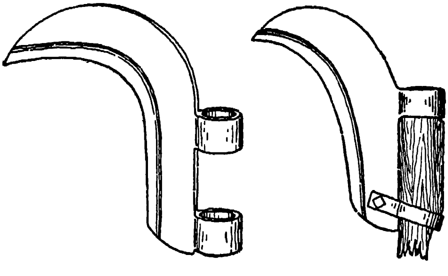


Fig. 38. Bush Hooks

Plows. Plows are employed extensively in grading, special forms being manufactured for the purpose. They are

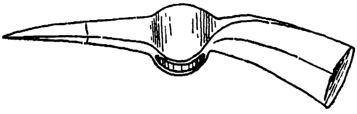


Fig. 39. Bush Mattock

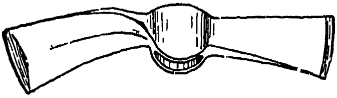


Fig. 40. Axe Mattock

They vary in form according to the kind of work they are intended for, viz, loosening earth, gravel, hardpan, and some of the softer rocks.

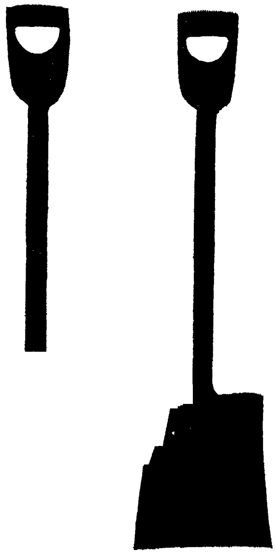


Fig. 41. Round Pointed and Square Shovels
Courtesy of Acme Road Machinery Company, Frankfort, New York

These plows are of great strength; selected white oak, rock elm, wrought steel, and iron generally being used in their construction. The cost of operating plows ranges from 2 to 5 cents per cubic yard, depending upon the compactness of the soil. The quantity of material loosened will vary from 2 to 5 cubic yards per hour.

Grading Plow. Fig. 42 shows the form usually adopted for loosening earth. This plow does not turn the soil, but cuts a furrow about 10 inches wide and of a depth adjustable up to 11 inches.

In light soil the plows are operated by 2 or 4 horses; in heavy soil as many as 8 are employed. Grading plows vary in weight from 100 to 325 pounds.

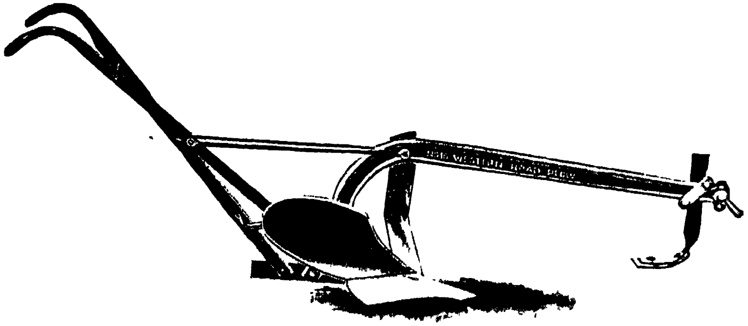


Fig 42 Typical Road Plow
Courtesy of Western Wheeled Scraper Company, Aurora, Illinois

Hardpan Plow. Fig. 43 illustrates a plow specially designed for tearing up macadam, gravel, or similar material. The point is a straight bar of cast steel drawn down to a point, and can be repaired easily

Scrapers. Scrapers are used generally to move the material loosened by plowing; they are made of either iron or steel, and in a variety of forms, and are known by various names, as "drag", "buck", "pole", and "wheeled". The drag scrapers are employed usually on short hauls, the wheeled ones on long hauls.

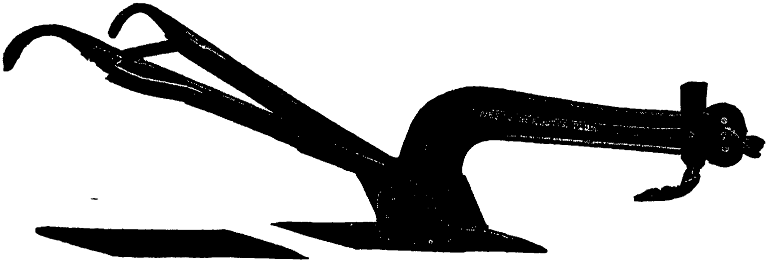


Fig. 43. Typical Hardpan or Rooter Plow
Courtesy Western Wheeled Scraper Company, Aurora, Illinois

Drag Scrapers. Drag scrapers, Fig. 44, are made in three sizes. The smallest, for one horse, has a capacity of 3 cubic feet; the others, for two horses, have a capacity of 5 to 7½ cubic feet. The smallest weighs about 90 pounds, and the larger ones from 94 to 102 pounds.

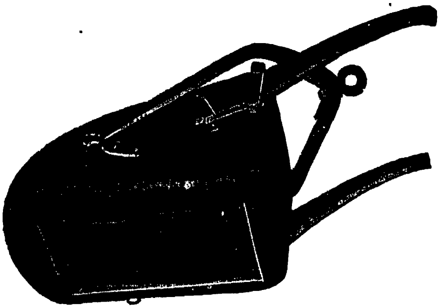


Fig 44. Drag Scraper
Courtesy Western Wheeled Scraper Company, Aurora, Illinois

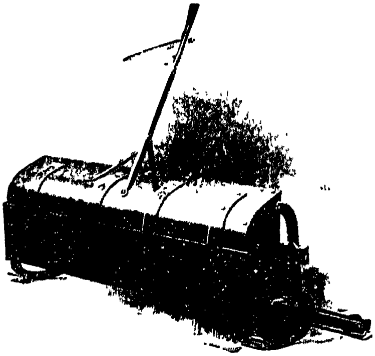


Fig 45 Buck Scraper

Buck Scrapers. Buck scrapers, Fig. 45, are made in two sizes—two-horse, carrying 7½ cubic feet; four-horse, 12 cubic feet.

Pole Scrapers. Pole scrapers are designed for use in making and leveling earth roads and for cutting and cleaning ditches; they are well adapted also for moving earth short

Wheeled Scrapers. Wheeled scrapers, Fig. 46, consist of a metal box, usually steel, mounted on wheels, and furnished with levers for raising, lowering, and dumping. They are operated in the same manner as drag scrapers, except that all the movements are made by means of the levers, and without stopping the team. By their use the excessive resistance to traction of the drag scraper is avoided.

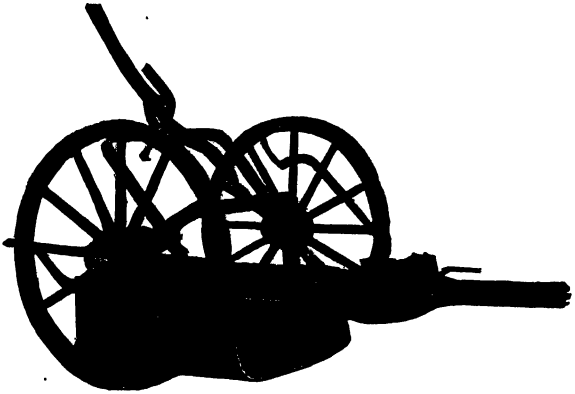


Fig. 46. Typical Wheeled Scraper
Courtesy Western Wheeled Scraper Company, Aurora, Illinois

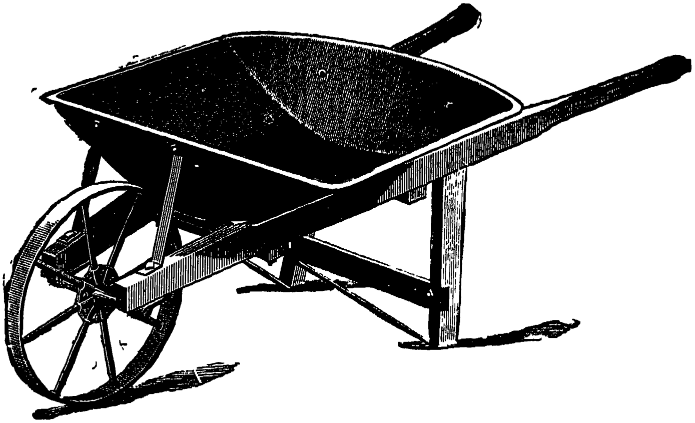


Fig. 47 Contractor's Barrow with Pressed-Steel Tray
Courtesy of Acme Road Machinery Company, Frankfort, New York

Various sizes are made, ranging in capacity from 10 to 17 cubic feet. In weight they range from 350 to 700 pounds.

Wheelbarrows. Wheelbarrows sometimes are constructed of wood and are employed most commonly [illegible] capacities range from 2 to $2\frac{1}{2}$ cubic feet. Weight is about 50 pounds.

The barrow, Fig. 47, has pressed-steel tray, oak frame, and steel wheels, and will be found more durable in the maintenance department than the all-wood barrow. Capacity is from $3\frac{1}{2}$ to 5 cubic feet, dependent on size of tray.

The barrow, Fig. 48, is constructed with tubular-iron frames and steel tray, and is adaptable to the heaviest work, such as moving heavy broken stone, etc., or it may be employed with advantage in the cleaning department. Capacity from 3 to 4 cubic feet. Weight from 70 to 82 pounds.

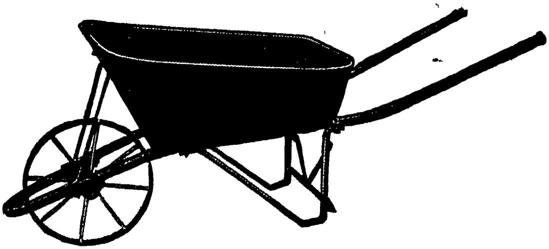


Fig. 48. All Steel and Iron Concrete Barrow
Courtesy of Acme Road Machinery Company, Frankfort, New York

The maximum distance to which earth can be moved economically in barrows is about 200 feet. The wheeling should be performed upon planks, whose steepest inclination should not exceed 1 in 12. The force required to move a barrow on a plank is about $\frac{1}{25}$ part of the weight; on hard dry earth, about $\frac{1}{14}$ part of the weight.

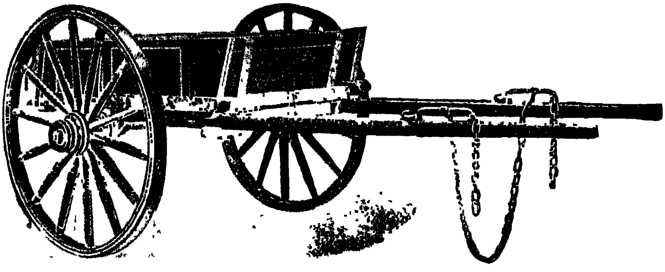


Fig. 49. Typical Dump Carts for Hauling Earth, Etc
Courtesy of Western Wheeled Scraper Company, Aurora, Illinois

The time occupied in loading a barrow will vary with the character of the material and the proportion of wheelers to shovel-with earth as a wheeler takes to wheel a full barrow a distance of about 100 or 120 feet on a horizontal plank and return with the empty barrow.

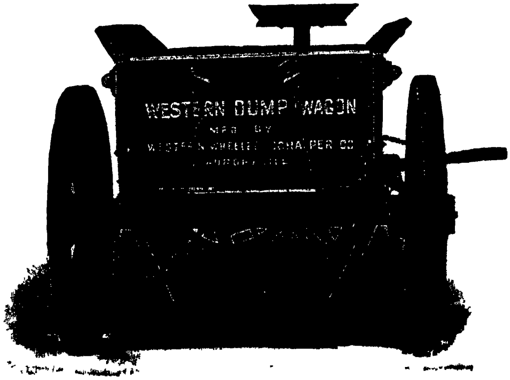


Fig. 50. Rear View of Typical Dump Wagon Showing Bottom Open
Courtesy of Western Wheeled Scraper Company, Aurora, Illinois

Carts. The cart usually employed for hauling earth, etc., is shown in Fig. 49. The average capacity is 22 cubic feet, and the average weight is 800 pounds. These carts are furnished usually with broad tires, and the body is balanced so that the load is evenly divided about the axle.

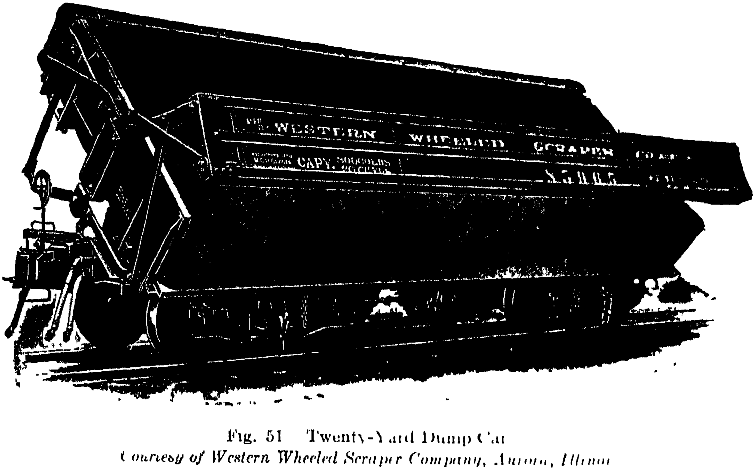


Fig. 51 Twenty-Yard Dump Car
Courtesy of Western Wheeled Scraper Company, Aurora, Illinois

The time required to load a cart varies with the material. One shoveler will require about as follows: clay, 7 minutes; loam, 6 minutes; sand, 5 minutes

Dump Wagons. The use of dump wagons, Fig. 50, for moving excavated earth, etc., and for transporting materials such as sand, gravel, etc., materially shortens the time required for unloading the ordinary form of contractor's wagon; having no reach or pole connecting the rear axle with the center bearing of the front axle, they may be cramped short and the load deposited just where required. They are operated by the driver, and the capacity ranges from 35 to 45 cubic feet.

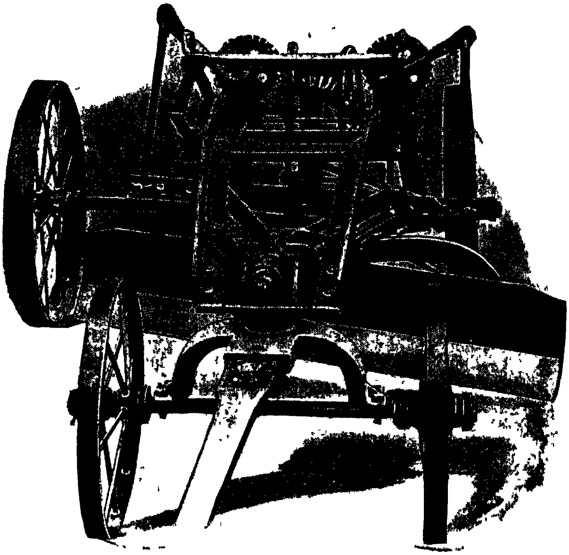


Fig 52 Typical Grader
Courtesy of Acme Road Machinery Company, Frankfort, New York

Dump Cars. Dump cars, Fig. 51, are made to dump in several different ways, viz, single or double side, single or double end, and rotary or universal dumpers.

Dump cars may be operated singly or in trains, as the magnitude of the work may demand. They may be moved by horses or small locomotives. They are made in various sizes, depending upon the gage of the track on which they are run. A common gage is 20 inches, but it varies from that up to the standard railroad gage of 56½ inches.

Mechanical Graders. Mechanical graders are used extensively in the making and maintaining of earth roads. They excavate

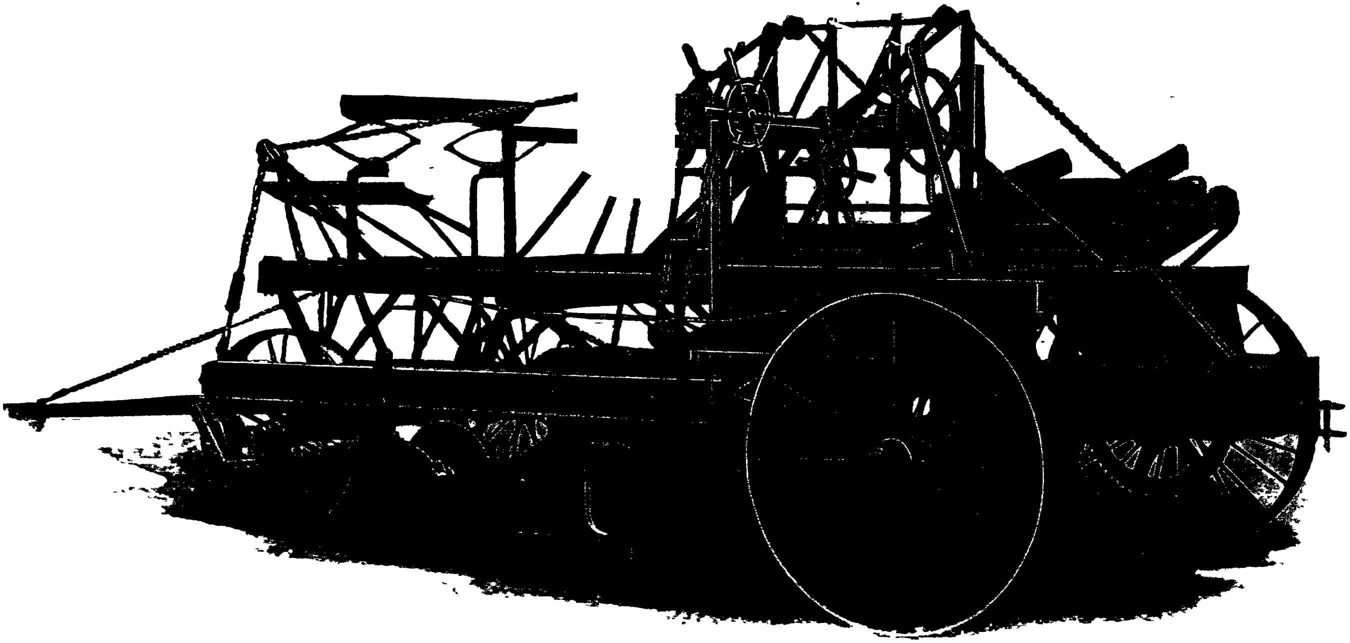


Fig. 53. Standard Elevating Grader

Courtesy of Western Wheeled Scraper Company, Aurora, Illinois done by hand; they are called by various names, such as "road machines", "graders", "road hones", etc.

Simple Graders. Briefly described, graders consist of a large blade, Fig. 52, made entirely of steel, iron, or wood shod with steel, which is so arranged by a mechanism attached to the frame from which it is suspended that it can be adjusted and fixed in any direction by the operator. In their action they combine the work of excavating and transporting the earth. They have been employed chiefly in the forming and maintenance of earth roads, but also may be used advantageously in preparing the subgrade surface of roads for the reception of broken stone or other improved covering.

Elevating Graders. Some graders combine the function of elevating the material, of excavating it from side ditches, and of loading it automatically into carts or wagons. Briefly described, the machine, Fig. 53, consists of a plow which loosens and raises the earth, depositing it upon a transverse carrying belt, which conveys it from excavation to embankment. Carrier frames of two or three different lengths are provided with the machine, the distance of the end of the elevator from the plow varying from 15 to 30 feet. The carrier belt is of heavy 3-ply rubber 3 feet wide.

The plow and carrier are supported by a strong trussed framework resting on heavy steel axles and broad wheels. The large rear wheels are ratcheted upon the axle, and connected with strong gearing which propels the carrying belt at right angles to the direction in which the machine is moving.

The wheels and trusses are low and broad, occupying a space 8 feet wide and 14 feet long, exclusive of the side carrier. This enables it to work on hillsides where any wheeled implements can be used. Notwithstanding its large size it is so flexible that it may be turned around on a 16-foot embankment. Pilot wheels and levers enable the operator to raise or lower the plow or carrier at pleasure.

For motive power, 12 horses—8 driven in front, 4 abreast, and 4 in the rear on a push cart—are usually employed.

When the teams are started, the operator lowers the plow and throws the belting into gear, and as the plow raises and turns the which the carrier is adjusted, forming either excavation or embankment, as the case may be.

When it becomes necessary to deliver the excavated earth beyond the capacity of the machine, the earth is loaded upon wagons,

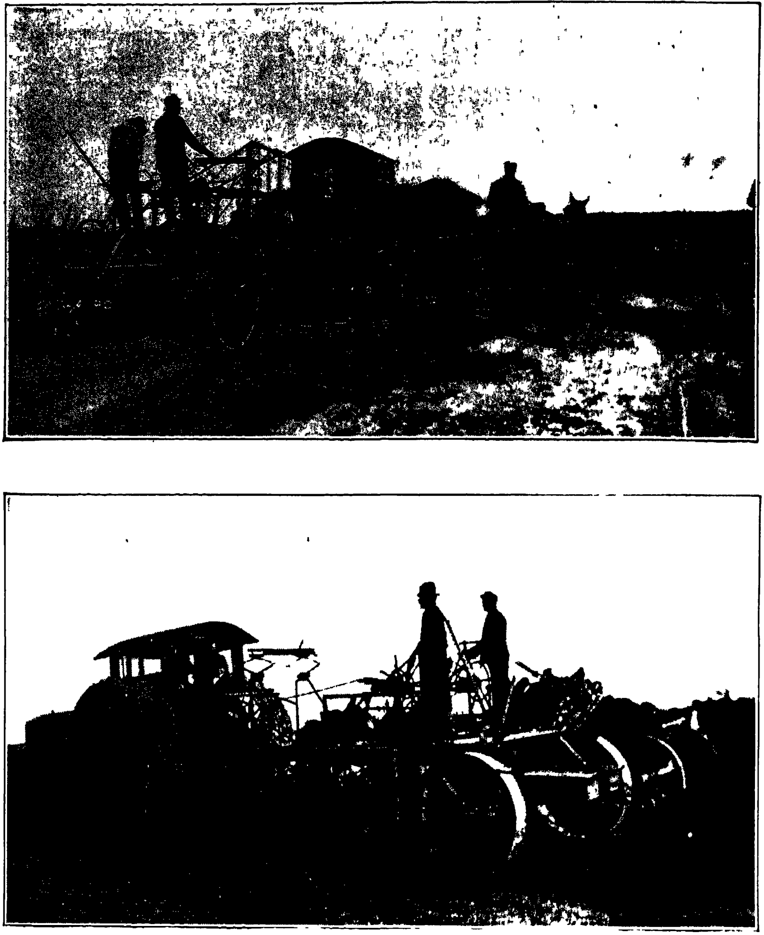


Fig. 54 Two Views of Elevating Graders Loading Earth into Dump Wagons
Courtesy of Western Wheeled Scraper Company, Aurora, Illinois

then conveyed to any distance. By adjusting the height of the carrier, the wagons are driven under it, Fig. 54, and loaded with $1\frac{1}{4}$ to $1\frac{1}{2}$ yards of earth in from 20 to 30 seconds. When one wagon

machine thus loads 600 to 800 wagons per day. It is claimed that with six teams and three men it is capable of excavating and placing in embankment from 1,000 to 1,500 cubic yards of earth in 10 hours, or of loading from 600 to 800 wagons in the same time, and that the cost of this handling is from 1½ to 2½ cents per cubic yard.

Points to be Considered in Selecting a Road Machine. In the selection of a road machine the following points should be carefully considered: thoroughness and simplicity of its mechanical construction; material and workmanship used in its construction; safety to the operator; ease of operation; lightness of draft; and adaptability to general road work, ditching, etc.

Care of Road Machines. The road machine when not in use should be stored in a dry house and thoroughly cleaned, its blade brushed clean from all accumulations of mud, wiped thoroughly dry, and well covered with grease or crude oil. The axles, journals, and wearing parts should be kept well oiled when in use, and an extra blade should be kept on hand to avoid stopping the machine while the dulled one is being sharpened.

Surface Graders. The surface grader is used for removing earth previously loosened by a plow. It is operated by one horse.

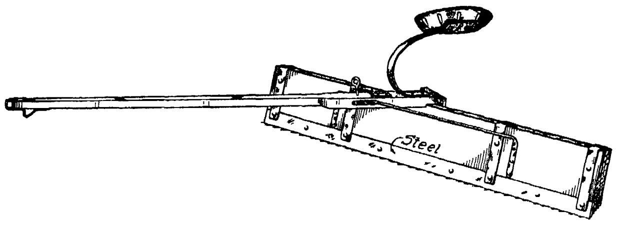


Fig. 55. Simple Road Leveler

The load may be retained and carried a considerable distance, or it may be spread gradually as the operator desires. It is also employed to level off and trim the surface following the scrapers.

The blade is of steel, ¼ inch thick, 15 inches wide, and 30 inches long. The beam and other parts are of oak and iron. Weight about 60 pounds.

Road Leveler. The road leveler, Fig. 55, is used for trimming and smoothing the surface of earth roads. It is largely employed in

The blade is of steel, $\frac{1}{4}$-inch thick by 4 inches by 72 inches, and is provided with a seat for the driver. It is operated by a team of horses. Weight about 150 pounds.

Ditching Tools. The tools employed for digging the ditches and shaping the bottom to fit the drain tiles are shown in Fig. 56. They are convenient to use, and expedite the work by avoiding unnecessary excavation.

The tools are used as follows: Nos. 3, 4, and 5 are used for digging the ditches; Nos. 6 and 7 for cleaning and rounding the bottom of the ditch for round tile; No. 2 is used for shoveling out loose earth and leveling the bottom of the ditch; No. 1 is used for the same purpose when the ditch is intended for "sole" tile.

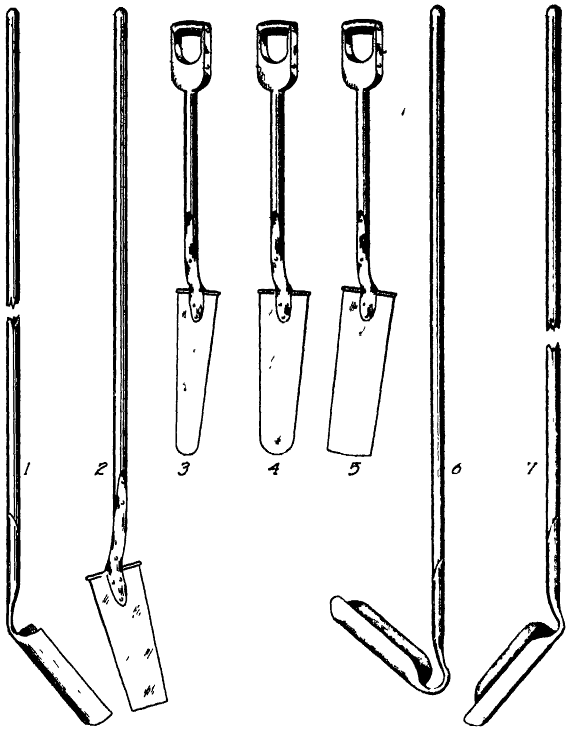


Fig. 56 Typical Tools Used for Digging Ditches

Sprinkling Wagons. A convenient form of sprinkling wagon for suburban streets and country roads is shown in Fig. 57. The tank is of 12 gage steel and its capacity is 380 to 600 gallons.

Road Rollers. *Horse-Drawn Rollers.* There are a number of types of horse-drawn rollers on the market, consisting essentially of a hollow cast-iron cylinder 4 to 5 feet long, 5 to 6 feet in diameter, and weighing from 3 to 6 tons. Some forms are provided with boxes in which stone or iron may be placed to

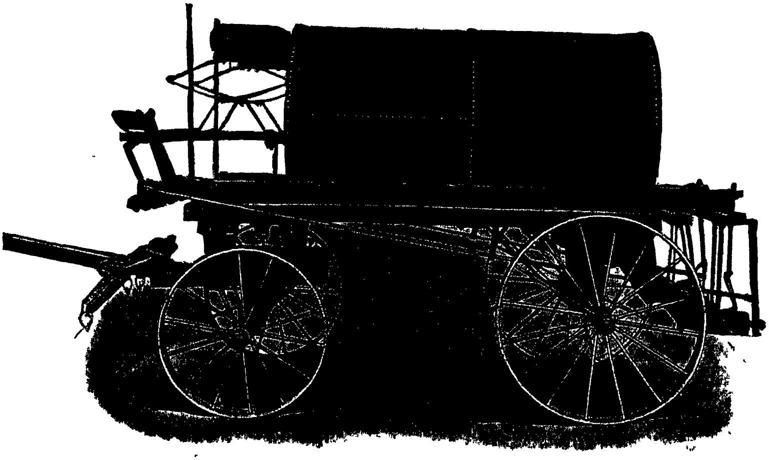


Fig. 57. Steel Tank Sprinkling Wagon
Courtesy of Acme Road Machinery Company, Frankfort, New York

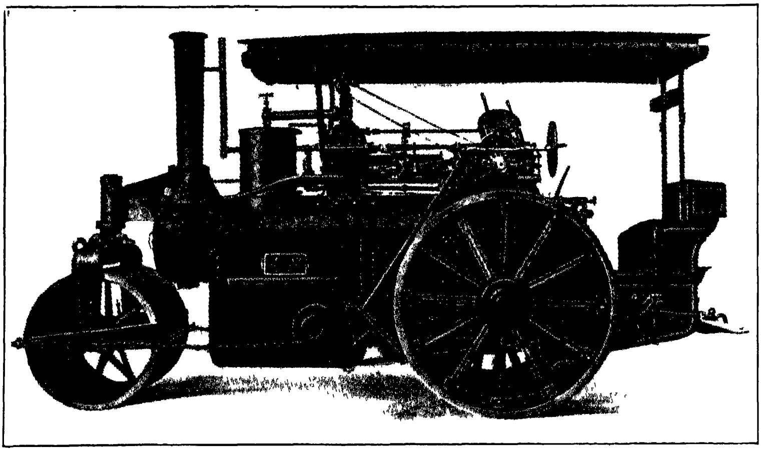


Fig 58. Ten-Ton Steam-Driven Road Roller
Courtesy of Charles Longenecker & Company, New York City

with water or sand. The use today of small gasoline road rollers makes this type less prevalent than formerly.

Power-Propelled Rollers. The rollers employed for compacting the natural soil and all forms of broken-stone pavements usually are of the three-wheel type, operated by steam or gasoline, Fig. 58. They generally are arranged to move at two speeds, low and high; the low speed is from 2 to 3 miles per hour and the high speed from 4 to 5 miles. The low speed is employed for compacting the natural soil and the foundation; the high speed is employed for finishing the surface. The driving wheels are furnished with lock pins or differential gears to permit them to accommodate themselves automatically to the difference in speeds when operating on sharp curves. They vary in weight from 10 to 20 tons.

Scarifiers. The implement used for breaking up a broken-stone road preparatory to applying a new surface is called a "scarifier", Fig. 59. It usually consists of a cast-iron block, weighing about 3 tons, mounted on 2 or 4 wheels; the block is fitted with a series of spikes or picks, arranged either in one line, or in two lines forming a V; means are provided for adjusting the depth to which the picks penetrate, the maximum depth being about 6 inches. The scarifier is operated by being attached to the rear of a steam roller or traction engine which hauls it over the road.

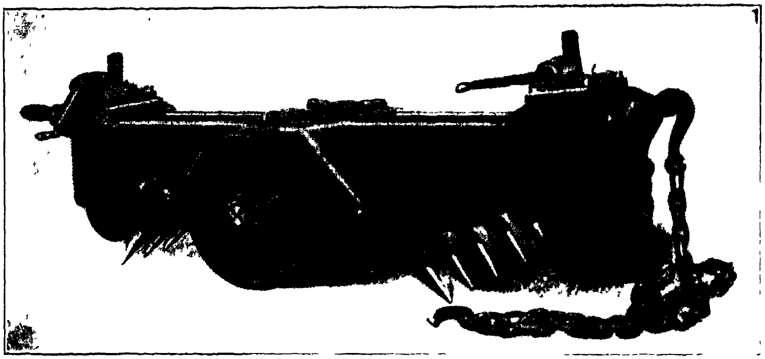


Fig 59. Scarifier, for Quick and Economical Repair of Macadam Roads
Courtesy of Charles Longenecker & Company, New York City

NATURAL-SOIL ROADS

Earth Roads. The term "earth road" is applied to roads where the surface consists of the native soil; this class of road is the most common and cheapest in first cost. At certain seasons of the year earth roads, when properly cared for, are second to none, but during the spring and wet seasons they are very deficient in the important requisite of hardness, and are almost impassable.

For the construction of new earth roads, all the principles previously discussed relating to alignment, grades, drainage, width, etc., should be followed carefully. The crown or transverse contour should be greater than in stone roads; 12 inches at the center in 25 feet will be sufficient.

Drainage is especially important, because the material of the road is more susceptible to the action of water, and more easily destroyed by it than are the materials used in the construction of the better class of roads. When water is allowed to stand upon the road, the earth is softened, the wagon wheels penetrate it, and the horses' feet mix and knead it until it becomes impassable mud. The action of frost is also apt to be disastrous upon the more permeable surface of the earth road, having the effect of swelling and heaving the roadway and throwing its surface out of shape. It may in fact be said that the whole problem of the improvement and maintenance of ordinary country roads is one of drainage.

In the preparation of the wheelway all stumps, brush, vegetable matter, rocks, and boulders should be removed from the surface and the resulting holes filled in with clean earth. The roadbed, having been brought to the required grade and crown, should be thoroughly rolled; all inequalities appearing during the rolling should be filled up and re-rolled.

Care of Earth Roads. If the surface of the roadway is properly formed and kept smooth, the water will be shed into the side ditches and do comparatively little harm; but if it remains upon the surface, it will be absorbed and convert the road into mud. All ruts and depressions should be filled up as soon as they appear. Repairs should be attended to particularly in the spring. At that season the judicious use of a road machine and rollers will make a smooth road. In summer when the surface gets rough it can be improved by running a harrow over it; if the surface is a little muddy this treatment will hasten the drying.

During the fall the surface should be repaired, with special reference to putting it in shape to withstand the ravages of winter.

Saucer-like depressions and ruts should be filled up with clean earth similar to that of the roadbed and tamped into place.

The side ditches should be examined in the fall to see that they are free from dead weeds and grass, and late in winter they should be examined again to see that they are not clogged. The mouths of culverts should be cleaned of rubbish and the outlet of tile drains opened. Attention to the side ditches will prevent overflow and washing of the roadway, and also will prevent the formation of ponds at the roadside and the consequent saturation of the roadbed.

Holes and ruts should not be filled with stone, bricks, gravel, or other material harder than the earth of the roadway as the hard material will not wear uniformly with the rest of the road, but produce bumps and ridges, and usually result in making two holes, each larger than the original one. It is bad practice to cut a gutter from a hole to drain it to the side of the road. Filling is the proper course, whether the hole is dry or contains mud.

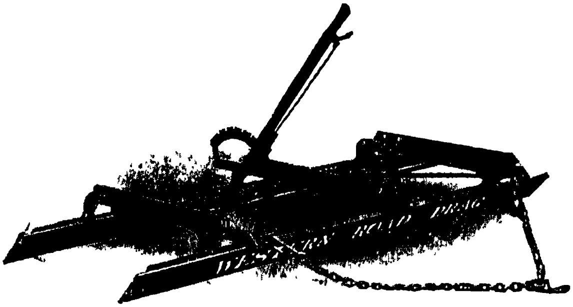


Fig. 60. Steel Road Drag
Courtesy of Western Wheeled Scraper Company, Aurora, Illinois

The maintaining of smooth surfaces on all classes of earth roads will be assisted and cheapened greatly by the frequent use of a roller (either steam or horse) and any one of the various forms of road grading and scraping machines. In repairing an earth road the plow should not be used. It breaks up the surface which has been compacted by time and travel.

In the maintenance of earth roads the road drag, Fig. 60, or some similar device, is indispensable. The drag should be light and should be hauled along the road at an angle of about 45 degrees, so that only a small road. The driver should ride on the drag and not drive faster than a walk. Dragging should begin on the side of the road, or wheel track, and return on the opposite side. Unless the road is in good condition, it should be dragged after every heavy rain.

In the maintenance of clay roads neither sods nor turf should be used to fill holes or ruts; for, though at first deceptively tough, they soon decay and form the softest mud. Neither should the ruts be filled with field stones; they will not wear uniformly with the rest of the road, but will produce hard ridges.

Trees and close hedges should not be allowed within 200 feet of a clay road. It requires all the sun and wind possible to keep its surface in a dry and hard condition.

Sand Roads. The aim in the improvement of sand roads is to have the wheelway as narrow and well defined as possible, so as to have all vehicles run in the same track. An abundant growth of vegetation should be encouraged on each side of the wheelway, for by this means the shearing of the sand is, in a great measure, avoided. Ditching beyond a slight depth to carry away the rain water is not desirable, for it tends to hasten the drying of the sands, which is to be avoided. Where possible the roads should be overhung with trees, the leaves and twigs of which, catching on the wheelway, will serve still further to diminish the effect of the wheels in moving the sands about. If clay can be obtained, a coating 6 inches thick will be found a most efficient and economical improvement. A coating of 4 inches of loose straw will, after a few days' travel, grind into the sand and become as hard and firm as a dry clay road.

Sand=Clay Roads. A sand-clay road is formed by mixing clay and sand in such proportions that the clay will just fill the voids in the sand, and produce a mixture that is neither sticky nor friable, but coheres in a comparatively dry plastic mass when compacted with pressure. If an insufficient amount of clay is used, the mixture will not bind; if an excess of clay is used, the road will be sticky and muddy after a fall of rain.

The grains of sand furnish the hard material to resist the abrasion of the traffic; the clay provides the cementing or binding medium to hold the sand together. All clays are not equally satistest for clay suitable for road purposes is to apply a wet finger to a piece of clay; if the clay adheres to the finger, it may be assumed reasonably that it will adhere to the sand.

The natural sand soils and the naturel clay soils are improved by the application of the sand-clay mixture, the method of applying it being varied according to which kind of soil is to be treated.

Applying Sand-Clay Mixture to Clay Soil. In the treatment of a clay soil, the soil is plowed to a depth of 6 to 8 inches; then pulverized by harrowing, and, if necessary, by rolling with a light roller and again harrowing. After the clay is thoroughly pulverized, the sand is spread over the surface in a layer from 6 to 8 inches thick, and the sand and clay are thoroughly mixed by continued harrowing. After the dry mixing has been accomplished satisfactorily, the surface is moistened slightly by sprinkling with water, then compacted by rolling, after which a road machine or grader is used to give the required crown; and then the roller is again applied the surface becomes smooth and hard.

Applying Sand-Clay Mixture to Sand Soil. In the treatment of a sand soil, the clay is spread over the surface in a layer, ranging from 4 to 8 inches thick; then mixed with the sand by harrowing. After that it is sprinkled heavily with water and again worked with the harrow; then it is shaped and rolled in the same manner as stated above for a clay soil.

The sand-clay roads require considerable attention, after completion, to eliminate weak or defective spots by applying sand or clay, as may be required.

Application of Oil to Sand and Gravel Soils. Sand and gravel soils are improved by the application of crude petroleum or asphaltic oils. The oil abates dust; forms a non-absorbent surface which turns off rain water and decreases the amount of mud; and furnishes a dark-colored road surface which is more pleasing to the eye than the ordinary light color.

The roadbed is prepared to receive the oil by grading, shaping, and rolling. The oil is applied to the prepared surface by sprinkling from tank wagons; the oil coat is covered with a thin layer of sand, after which the roller is applied again. If during the rolling the surface becomes sticky, or dry and dusty, dry sand or more oil is added as required.

ROADS WITH SPECIAL COVERINGS

Elements of a Road Covering. The wheelways of roads and streets are prepared for traffic by placing upon the natural soil a covering of some suitable material which will furnish a comparatively smooth surface on which the resistance to traction will be reduced to the least possible amount, and over which all classes of vehicles may pass with safety and expedition at all seasons of the year. The covering usually consists of two parts: a foundation, and a wearing surface.

The functions of the foundation are as follows: (1) to protect the soil from disturbance and the injurious effects of surface water; (2) to transmit to and distribute over a sufficiently large area of the soil the weight of the loads imposed upon the wearing coat; (3) to support unyieldingly the wearing surface and the loads coming upon it.

The efficiency of the wearing surface depends entirely upon the quality of the foundation. If the foundation be weak, the wearing surface will be disrupted speedily, no matter how well constructed.

FOUNDATIONS

Materials. The foundation, when once constructed, should not require to be disturbed nor reconstructed. The materials employed in its construction may be the cheapest available, such as local rock, gravel, sand, furnace slag, etc., the important point in the design being to provide sufficient thickness, so that when consolidated it will maintain its form under the heaviest traffic liable to come upon it. If the foundation and the covering yield under the load, an upheaval is caused that disrupts the bond and hastens the destruction of the road.

Thickness. The thickness of the foundation depends upon the supporting power of the natural soil and the weight of the loads coming upon the wearing surface. The supporting or bearing power of the soil can be ascertained by direct test, and the weight of the loads by a survey of the traffic plus a provision for future increase.

Recent tests indicate that non-porous soils from which the condition a load of about 4 pounds per square inch; and that if the thickness of the foundation be adjusted to the traffic on this basis it will be safe at all seasons of the year.

Methods of Calculating Thickness of Covering. There are two theories as to the manner in which pressure of a loaded wheel is transmitted from the surface of the covering to the natural soil: (1) that the pressure on the soil varies inversely as the cube of the thickness of the foundation and the wearing surface; (2) that the pressure is transmitted downwards in the form of a truncated cone, the lines of which diverge at an angle varying from 30 to 50 degrees from the vertical, according to the solidity of the covering. If the surface of the road is uneven or obstructed by loose stones, the lines of pressure are more concentrated when the wheels pass over such obstacles.

The latter theory is the one most frequently applied. The calculation is performed as follows: Let P be distributed pressure on the soil, per square inch; A, length of arc of wheel tire in contact with surface in square inches; W, width of tire in inches; L, load carried by wheel in pounds; F, depth of wearing surface and foundation in inches; C, area of contact equal to $A \times W$; and B, area of base at surface of natural soil. The area of the base is

$$B = (2F + A)\ (2F + W)$$

The distributed pressure is

$$P = \frac{L}{(2F + A)\ (2F + W)} = \frac{L}{B}$$

Assuming that the load is 1,000 pounds per inch of tire width; the tire, 3 inches; length of contact 3 inches; total thickness of the wearing coat and the foundation 12 inches; the pressure on the soil is

$$P = \frac{1000 \times 3}{(2 \times 12 + 3)\ (2 \times 12 + 3)} = \frac{3000}{27 \times 27} = \frac{3000}{729} = 4.10 \text{ lb. per sq in.}$$

According to this theory the thickness of the covering varies from 4 to 16 inches, the smallest thickness being placed upon gravel or sand and the greatest upon clay.

Preparation of Foundation. The preparation of the foundation involves two distinct operations: (1) preparation of the natural soil; and (2) placing an artificial foundation upon the prepared natural soil

The essentials necessary to the preparation of the natural soil are: (1) the entire removal of perishable vegetable and yielding matter; (2) the drainage of the soil where necessary; (3) the improving of the bearing power of the soil where required; and (4) compacting the soil.

All soils are improved by rolling, and weak spots, which otherwise would pass unnoticed, are discovered. However, care must be taken that the weight of the roller employed is not too great for the bearing power of the soil; if it exceed this the surface of the soil will be formed into a series of undulations that will cause the wearing coat to fail; the same condition may be produced by excessive rolling with a comparatively light roller. Each soil requires different treatment.

Soils of a siliceous and calcareous nature may be improved by drainage and the addition of a layer of clay 2 to 6 inches thick, mixed with the soil and compacted by rolling. The argillaceous and allied soils, owing to their retentive nature, are very unstable under the action of water and frost, and in their natural condition afford a defective foundation. They are improved by thorough drainage and the admixture of sand well rolled, together with the placing upon the surface of the compacted soil a layer 2 to 6 inches thick of sand, slag, cinders, or other material of a similar nature, and then compacting it by sprinkling with water and rolling.

Types of Foundation to Be Used. The essential requisite in the construction of the artificial foundation is that it be a dense mass, and the type of foundation to be employed varies with the character of the wearing surface. For the various types of broken-stone surfaces, the foundation may be composed of blocks of stone (ledge rock or fieldstones), roughly shaped to a rectangular form, ranging in width and depth from 6 to 8 inches and in length from 6 to 16 inches. They are set by hand on the soil bed with the length at right angles to the axis of the roadway, so arranged that they break joints. The edges that project above the subgrade level are broken off with hand hammers, and the spaces between them are filled with chips of stone well packed and wedged in. The blocks are brought then to a firm bearing by rolling with a steam roller, after which the wearing surface is laid. The foundation also may the voids will be reduced to the smallest possible amount. The voids may be filled with stone dust; a mixture of sand and clay; a mortar and grout composed of hydraulic cement and sand; bituminous cement; or hydraulic-cement concrete, mixed and placed upon the soil bed.

WEARING SURFACES

Functions of Wearing Surface. The office of the wearing surface is to protect the foundation from the wear of the traffic and the effects of surface water, and to support the weight of the traffic and transmit it to the foundation. To render efficient service to the traffic, it must furnish a comparatively smooth unyielding surface that affords good foothold for draft animals and good adhesion for motor vehicles, and on which the resistance to traction will be a minimum. To fill its office satisfactorily the material of which it is composed must possess strength to resist crushing and abrasion, and its fabric must be practically impervious. To render economical service, it must possess the power of resisting the action of the destroying agencies for a reasonable length of time before it becomes unfit for use. For this purpose it must possess the resisting qualities previously stated, and it must also possess a certain thickness; this thickness will depend upon the character of the material employed and its rate of wear under the given traffic and atmospheric conditions. Economy is not promoted by using a thick wearing surface, as under heavy traffic it will be so worn in a few years as to be unserviceable, and under light traffic it will be decomposed before it is worn out. In either case it must be removed and the portion so removed is waste; therefore, only such thickness as will give efficient service during a few years should be adopted.

Thickness. The measure for the economical thickness of any type of wearing surface is that the annual interest charge on the first cost plus the annual depreciation shall be a minimum. To apply this measure it is necessary to know the amount of traffic and the loss of thickness due to wear.

Classification of Wearing Surfaces. The wearing surfaces most commonly employed for roads and streets are composed of: (1) gravel, broken stone, furnace slag, and similar granular materials bound with colloidal cement formed by the action of water on the plastic elements of rock and clay; (2) broken stone, gravel, and sand bound with: (a) bituminous cement; (b) hydraulic cement; (3) stone blocks; (4) brick; (5) wood blocks.

In type (1), a certain amount of moisture is essential to successful binding. When this is lacking, as in the summer season, the binding material becomes dry and brittle, and the fragments at the surface are displaced by the action of the traffic; an excess of moisture destroys the binding power; and the surface is quickly broken up by the traffic.

Wearing surfaces of type (2a) are usually limited in life not merely by the wear of traffic, but by the fact that all bitumens slowly alter in chemical composition when exposed to atmospheric action, and in time become brittle. Type (2b) is subject to cracking under expansion and contracting, due to changes of temperature, and is liable to wear unevenly owing to irregularity in mixing and the segregation of the ingredients while the concrete is being put in place. When a defective spot begins to wear, it extends very rapidly under the abrasive action of the traffic.

The materials of types (3) and (4) seldom rot or disintegrate and, when the pavement is well constructed, are eminently enduring and generally render satisfactory service. Since the use of creosote and other preservatives has increased the service life of wood blocks, type (5), by lessening their tendency to decay, they have come into extensive use for street paving.

Gravel Roads

Gravel. Gravel consists of smooth and somewhat rounded stones, varying in size from small grains to pebbles 4 or more inches in diameter. It is found mixed with sand, on the banks and in the beds of rivers; and in deposits on the land, mixed with clay and other mineral substances, such as limestone and oxide of iron, from which it derives a distinctive name. Gravel of the latter class is called cementatious and when suitably prepared cements together, forming a very satisfactory roadway for light traffic, producing but little dust in dry weather and costing little to maintain.

Preparation of Gravel. Gravel is best prepared for use by screening into three grades: grade (1), containing the stones retained by a 1½-inch mesh screen and passing a 2½-inch mesh; grade (2), the voids will be reduced to the smallest possible amount. The voids may be filled with stone dust; a mixture of sand and clay; a mortar and grout composed of hydraulic cement and sand; bituminous cement; or hydraulic-cement concrete, mixed and placed upon the soil bed.

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Preparation of Gravel. Gravel is best prepared for use by screening into three grades: grade (1), containing the stones retained by a 1½-inch mesh screen and passing a 2½-inch mesh; grade (2), containing the stones retained by a ½-inch mesh and passing a 1½-inch mesh; grade (3), containing all the material passing the $\frac{1}{4}$-inch mesh. The voids in grade (1) are determined, and enough of grade (3) added slightly more than to fill them; the two are intimately and evenly mixed and the mixture is used for the first or lower course. The voids in grade (2) are determined and a sufficient quantity of grade (3) added to fill them; the two are mixed and used for the top course. The mixture should be combined very evenly so that the fine material is mixed uniformly with the coarse; and in spreading the mixture, care should be taken to avoid separating it or allowing the fine material to settle to the bottom.

If the gravel is deficient in binding material, the latter may be added in the form of clay, loam, limestone screenings, shale, or marl, the amount added ranging from 10 to 15 per cent. An excess (20 per cent) of clay causes the gravel to pack quickly and to present a good appearance under the rolling; but in dry weather the road will ravel, become defective and dusty, and in wet weather it will be muddy. Clean smooth gravel will not consolidate without a binder and. unless this is of very good quality, a road made with it will unsatisfactory.

Laying the Gravel. On the natural-soil bed properly graded and compacted, the prepared gravel is spread uniformly to the depth desired—usually 6 inches. Then it is compacted by rolling with a steam roller, after which it is moistened by sprinkling with water, and the rolling is repeated. The sprinkling and rolling are repeated as often as may be required, until the stones cease to rise or creep in front of the roller. The second course then is spread to a depth of about 4 inches, rolled, sprinkled, and again rolled in the same manner and to the same extent as the first course. After this, a thin coat of the fine screenings is spread over the surface and the traffic is admitted.

If, during the rolling, the first course appears to be deficient in binding material, more may be added by spreading a thin layer of the fine material over the surface of the course, sprinkling and rolling, as above described.

If, during the rolling of the top course, any stones larger than $1\frac{1}{2}$ inches appear, they must be removed.

Gravel shrinks in rolling about 20 per cent of its loose depth; therefore, to obtain a thickness of 8 inches when compacted, the loose material should have a depth of about 10 inches. The thickness of the gravel coating varies according to the nature of the roadbed, a thicker layer being necessary on impermeable soil than on a well-drained soil.

The pebbles in a gravel road are imbedded in a paste and can be displaced easily. It is for this reason, among others, that such roads are subject to internal destruction.

The binding power of clay depends in a large measure upon the state of the weather. During rainy periods a gravel road becomes soft and muddy, while in very dry weather the clay will contract and crack, thus releasing the pebbles, and causing a loose surface. The most favorable conditions are obtained in moderately damp or dry weather, during which a gravel road offers several advantages for light traffic, the character of the drainage, etc., largely determining durability, cost, maintenance, etc.

Repair. Gravel roads constructed as above described will need only small repairs for some years, but daily attention is required in making these. A garden rake should be kept at hand to draw any loose gravel into the wheel tracks, and for filling any depressions that may occur.

In making repairs, it is best to apply a small quantity of gravel at a time, unless it is a spot which actually has cut through. Two inches of gravel at once is more profitable than a larger amount. Where a thick coating is applied at once it does not all pack, and if, after the surface is solid, a cut be made, loose gravel will be found; this holds water and makes the road heave and become spouty under the action of frost. It will cost no more to apply 6 inches of gravel at three different times than to do it at once.

At every $\frac{1}{8}$ mile a few cubic yards of gravel should be stored to be used in filling depressions and ruts as fast as they appear, and there should be at least one laborer to every 5 miles of road.

Broken-Stone Roads

Methods of Construction. Broken-stone roads are formed in several different ways. For example, the road may be formed by placing one or two layers of stone broken into small fragments upon: (1) the natural soil; (2) a foundation composed of large stone

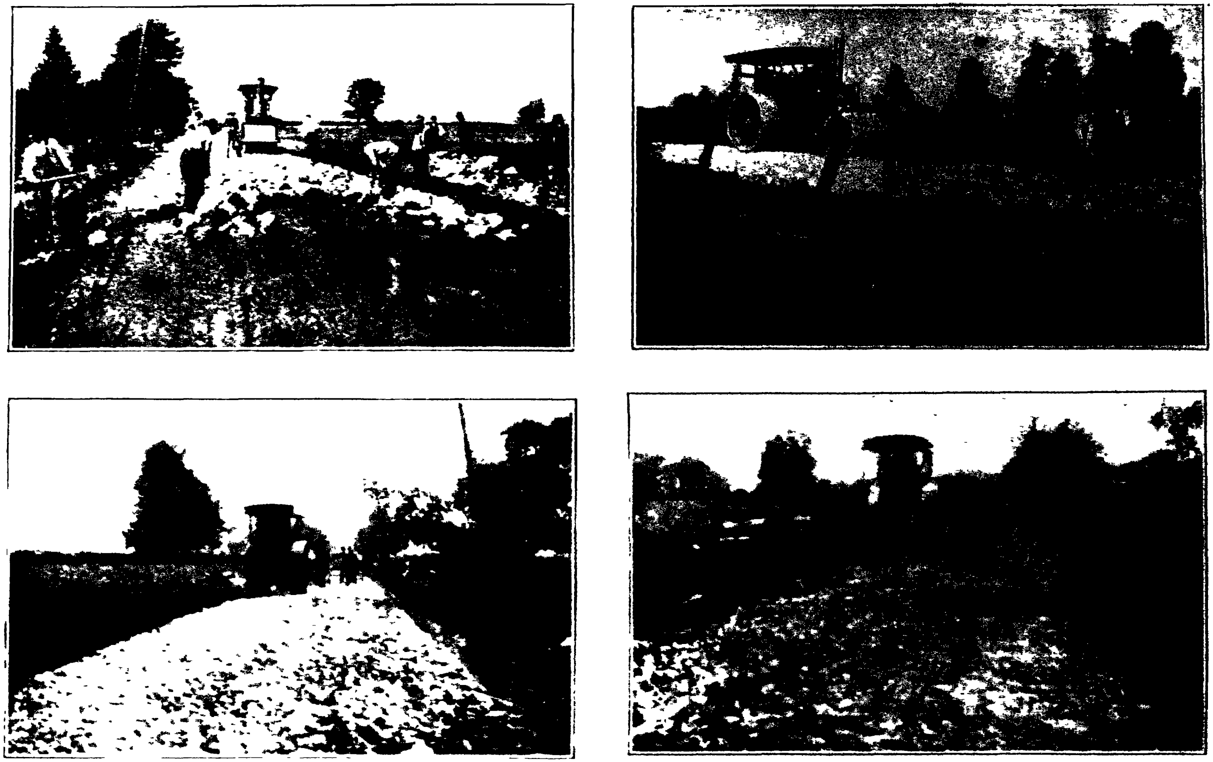


Fig. 61. Views Showing Method of Building Telford Roads for Pennsylvania State Highway Department
Courtesy of Charles Longnecker & Company, New York City

concrete. The layers of broken stone are compacted by rolling with a heavy roller and the interstices, or spaces between the stones are filled with a binder composed either of stone dust; stone dust and clay; a grout of hydraulic or Portland cement; or a bituminous cement derived from either coal tar or asphalt, and used alone or mixed with sand. The broken stone forming the lower surface layer often is coated with a bituminous cement before placing it upon the foundation. This applies particularly when the upper layer is of bituminous cement. The broken stone also may be mixed with Portland cement and sand, forming a concrete, which is placed either upon the prepared natural soil or upon a concrete or broken-stone foundation.

The several methods for constructing broken-stone roads are distinguished by either a specific name or the name of the introducer. Thus, the types known as *Telford* and *Macadam* are named from Thomas Telford and John L. McAdam, Scottish engineers, who introduced them in England during the early part of the 19th Century, as an improvement of the method employed in the 18th Century by M. Tresaguet on the roads of France. Telford used a base of large stones, Fig. 61, upon which the small stone was placed. McAdam omitted the base contending that it was useless and injurious. Both constructors insisted on thorough drainage of the subsoil, but neither used a binder and rolling was unknown. The stones were left to be compacted by the traffic. The introduction of stone-crushing machinery and rollers as well as the practice (condemned by McAdam, but advocated by Mr. Edgeworth, an Irish landowner in his treatise on Road Building published in 1817) of filling the voids with a binder has caused material departures from the methods of the pioneers whose names are still but improperly applied.

The cement grouting was introduced in England by Sir John Macneil. The coating of the stone with coal tar was first practiced in England about 1840, and was called "tar-macadam". In recent times, to distinguish the several varieties of bituminous construction, several specific terms have been coined, as "bitulithic", "tarmac", "warrenite", "bituminous macadam", "asphalt macadam", etc. Since the use of bituminous binding has become extensive, the term "water-bound macadam" has come into use, to distinguish the earlier macadam type from the types recently introduced.

Quality of Stones. The materials used for broken-stone pavements of necessity must vary very much according to the locality. Owing to the cost of haulage, local stone generally must be used, especially if the traffic be only moderate. If, however, the traffic is heavy, it sometimes will be found better and more economical to obtain a superior material, even at a higher cost, than the local stone; and in cases where the traffic is very great, the best material that can be obtained is the most economical.

There are a number of qualities required in a stone to render efficient service. *Hardness* and *toughness,* to resist the effects of abrasion and impact. These two properties, while closely related, are not always coincident; some rocks, although extremely hard, yet are so brittle that they crush easily under impact. In others the cohesion between the component particles is so weak that they are worn quickly by abrasion. *Durability,* or power to resist the disintegrating influences of the weather and humus acids. The quality of durability depends chiefly upon the chemical stability of the minerals present. Physical defects and abrasion generally cause the destruction of the stone long before it is injured by chemical changes. *Capability of binding* into a compact mass. This quality is essential to stone used for water-bound macadam. The binding or cementing property is possessed to a greater or less extent by all rocks when in a state of disintegration. It is caused by the action of water upon the chemical constituents of the stone contained in the detritus—material worn off—produced by crushing the stone, and by the friction of the fragments on each other while being compacted; its strength varies with the different species of rock, but it exists in some measure with them all, being greatest with limestone and least with gneiss.

The essential condition of the stone to produce this binding effect is that it be sound. No decayed stone retains the property of binding, though in some few cases, where the material contains iron oxides, it may, by the cementing property of the oxide, undergo a certain amount of binding.

A stone of good binding nature frequently will wear much better than one without, although it is not so hard. A limestone road well made and of good cross section will be more impervious than any other, owing to this cause, and will not disintegrate so soon in dry weather, owing partly to this and partly to the well-known quality which all limestone has of absorbing moisture from the atmosphere. Mere hardness without toughness is not of much use, as a stone may be very hard but so brittle as to be crushed to powder under a heavy load, while a stone not so hard but having a greater degree of toughness will be uninjured.

A stone for a road surface should be as little absorptive of moisture as possible in order that it may not suffer injury from the action of frost. Many limestones are objectionable on this account.

The stone used should be uniform in quality, otherwise it will wear unevenly, and depressions will appear where the softer material has been used. As the under parts of the road covering are not subject to the wear of traffic, and have only the weight of loads to sustain, it is not necessary that the stone of the lower layer be so hard or so tough as the stone for the surface, hence it is frequently possible by using an inferior stone for that portion of the work, to reduce greatly the cost of construction.

Testing the Rock. In order to ascertain the probable resistance of the different rocks to the destructive action of the traffic and weather, tests are made in the laboratory to determine the resistance to impact and abrasion, absorptive capacity, hardness, toughness, and specific gravity.

Abrasion. The test for abrasion is conducted in the Deval type of machine. It consists of two or more cast-iron cylinders mounted on a shaft so that the axis of each cylinder is inclined an angle of 30 degrees from the axis of rotation. The cylinders are charged with 11 pounds of the rock broken into fragments, ranging from $1\frac{1}{4}$ to $2\frac{1}{2}$ inches. The cylinders are then rotated at a uniform speed of 2,000 revolutions per hour for five hours, or until the automatic recorder shows 10,000 revolutions; the charge then is removed and placed on a sieve having meshes of $\frac{1}{16}$ inch. The material retained on the sieve is washed, dried, and weighed. The difference in weight between the weight of the charge and the residue larger than $\frac{1}{16}$ inch shows the loss by abrasion.

Impact and Toughness. The test for impact and toughness is made in a machine, consisting of an anvil, plunger, and hammer, mounted in vertical guides. The test piece is placed on the anvil; the hammer weighing 4.40 pounds is raised and allowed to fall a distance of one centimeter for the first blow and an increased fall of one centimeter for each succeeding blow, until the test piece fails. The number of blows required to destroy it is used to represent the toughness; 13 blows is considered to indicate low resistance, 13 to 19 medium, and above 19 high.

Hardness. The test for hardness is made on a Dory machine, which consists of a steel disk mounted so as to be revolved. The test pieces are cylinders cut from the rock by a core drill, and the ends ground level. Two pieces are used for a test; each is weighed, then placed in the guides of the machine with its face resting upon the grinding disk. The machine is revolved until 1000 revolutions have been made, and during the operation, quartz sand is fed onto the disk. The test piece is removed and weighed, and the hardness is determined from the formula

$$\text{Hardness} = 20 - \frac{W}{3}$$

in which W is loss in grams per 1000 revolutions. Rocks having a hardness less than 14 are considered soft; from 14 to 17 medium; and over 17 hard.

Water Absorption. The capacity of the stone to absorb water is determined by using a thoroughly dry sample of stone weighing about 12 grams. The sample is weighed in air, then immersed in water where it is weighed immediately; after 96 hours' immersion it is weighed again in the water. The absorptive capacity then is calculated by the formula

$$\text{Lb. water absorbed} = \frac{C - B}{A - B} \times 62.37 \text{ per cu. ft. of rock}$$

in which A is the weight in air; B is the weight in water immediately after immersion; C is the weight in water after immersion for 96 hours; and 62.37 is the normal weight in pounds of a cubic foot of water.

The durability of a stone used for roads is affected to a certain extent by its capability of absorbing water. In cold climates a low absorptive capacity is essential to resist the disintegrating effects of alternate freezing and thawing.

Specific Gravity. The specific gravity is determined either by weighing in a specific gravity balance or by weighing in air and water, and applying the formula

$$\text{Specific gravity} = \frac{W}{W - W_1},$$

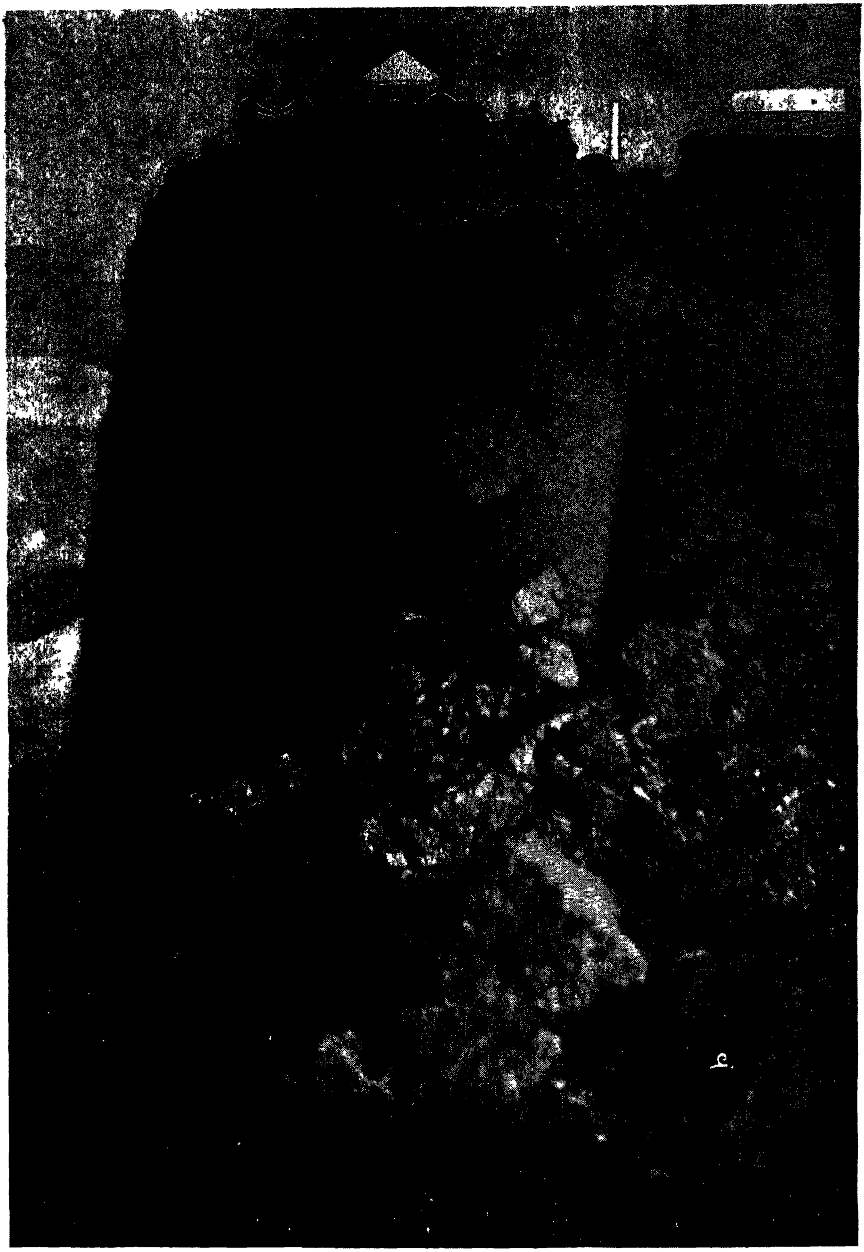
in which W is the weight in air, and W_1 is the weight in water.

Specific gravity and porosity are closely related. The specific gravity varies with the density or compactness of the aggregation of the mineral grains forming the stone. The closer the grains the more compact the stone, and the less will be the amount of interstitial space and hence the less the porosity.

From the specific gravity the weight per ton or per cubic yard may be determined. A knowledge of the weight is useful in deciding between two otherwise good stones; the heavier will be the more expensive, due to increased cost of transportation. On a water-bound macadam road it is an advantage to have a detritus with a high specific gravity, as it will not be moved so easily by rain and wind as one of low specific gravity.

Cementing Quality. The cementing quality of the stone dust is determined by placing 500 grams of the rock, broken to pass a ½-inch mesh screen, in a ball mill, together with 90 cubic centimeters of water and 2 steel balls weighing 20 pounds. The mill and its charge are revolved for 2½ hours at a rate of 2000 revolutions per hour. The operation produces a stiff dough, of which 25 grams are placed in a metal die 25 millimeters in diameter, and subjected to a pressure of 132 kilograms per square centimeter, producing a cylindrical test piece. The test piece is dried in the air for 20 hours, after which it is heated in a hot-air oven for 4 hours at a temperature of 200° Fahrenheit and then cooled in a desiccator for 20 minutes. When cool it is tested in the impact testing machine in the same manner as the test for toughness, using a hammer weighing 1 kilogram and a fixed height of fall of 1 centimeter. Blows are struck until the test piece fails. The average of the number of blows on 5 test pieces is taken as the result of the test. A result of 10 is considered to indicate a low cementing quality; 10 to 25 is considered fair; 26 to 75 good; 76 to 100 very good; over 100 excellent.

Species of Stone. The rocks most extensively used for broken-stone roads are trap, granite, limestone, sandstone, boulders, or field-



TRIANGULATION STATION ON MOUNT UNCOMPAHGRE. ELEVATION 14,300 FEET

Courtesy of U S Coast and Geodetic Survey, Washington, D C.

of the stone under the roller, the depth of the courses of loose stone should exceed the finished depth by from 25 to 30 per cent.

The stone is hauled upon the roadbed in vehicles of various types provided with broad-tired wheels. In some types of vehicle it is spread in layers as the vehicle is drawn along the roadbed; with others it is dumped in heaps and spread by hand with forks and brought to an even surface by raking, Fig. 62.

Compacting the Broken Stone. The stone is compacted by rolling with heavy rollers drawn by horses or propelled by steam or other power, Fig. 63. The steam roller is more effective than horse-drawn rollers. The usual weights of steam rollers are 5, 10, and 15 tons; the 10-ton being the one generally used, although the weight of the roller should be selected in accordance with the bearing power of the natural soil. A roller having excessive weight may cause injury to the roadbed, by rolling it into undulations that will permit water to collect and consequently cause damage. A roadbed which will stand a heavy roller in dry weather may be injured by it during wet weather. For a weak roadbed it is well to use two rollers, one of

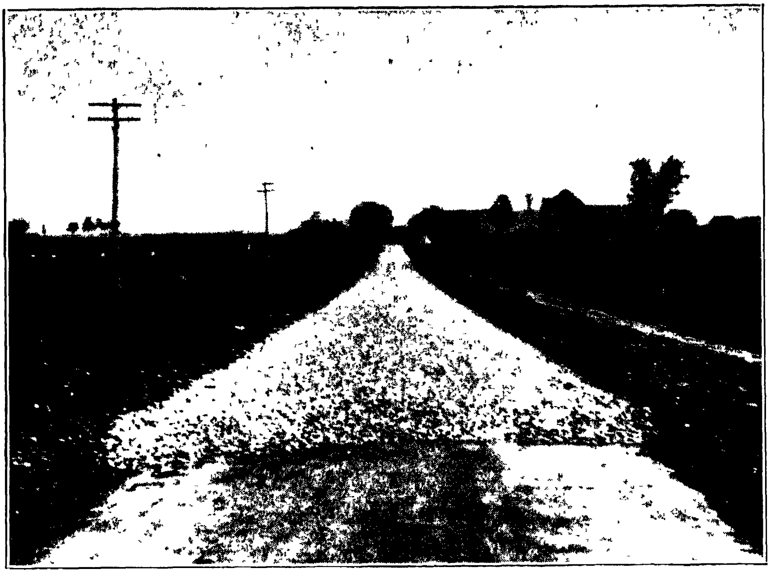


Fig. 62. View Showing Spreading of Lower Course of Macadam Road
Courtesy of United States Department of Agriculture

The roller should commence at one edge or border of the roadway, and move along that edge until within about 25 feet of one end of the spread stone; it then should cross over to the other edge and proceed along this edge to the beginning, crossing over and overlapping the strips previously rolled until the center of the road is reached. The rolling is continued in this manner until the stones cease to creep in front or sink under the roller. If, during the first passages of the roller, low spots appear, they should be filled to grade with stone of the same size as is in the course being rolled.

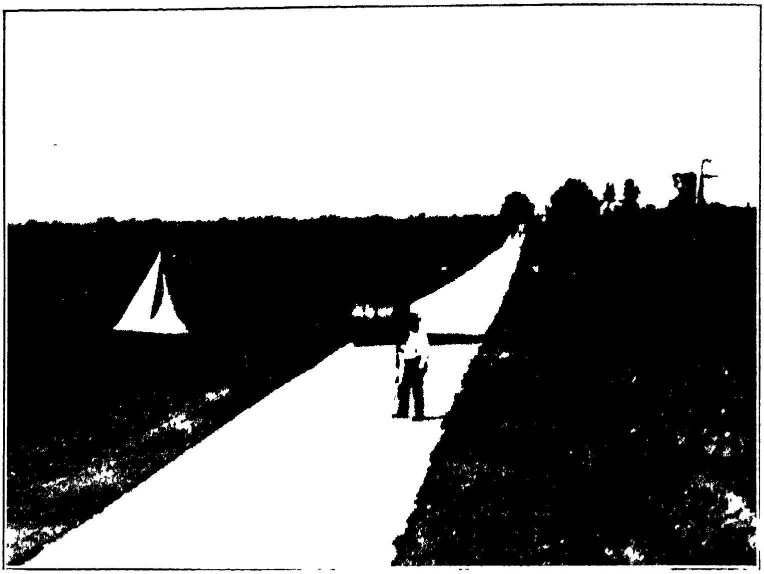


Fig 63 Compacting Broken Stone by Steam Roller
Courtesy of United States Department of Agriculture

After about two passages of the roller, the binder, consisting of the screenings from the stone being used for the course, is spread in a thin layer over the surface of the partly compacted stone and sprinkled with water, which washes it into the voids in the stone, the rolling then is continued, Fig. 64. The operation of applying the binder, sprinkling, and rolling is repeated until a wave of water and screenings rises in front of the roller. Each course is treated and rolled in the same manner. If the screenings from the rock that is being used are not suitable for binding, screenings from other

An excess of binder and water will shorten the time required to consolidate the stone and produce the appearance of a good piece of work, but under traffic it will wear unevenly and go to pieces quickly.

Suppression of Dust on Macadam and Telford Roads. Since the introduction of mechanically propelled vehicles, broken-stone roads constructed according to the principles of Telford and McAdam, have proven inadequate to the demands of the changed traffic.

The adhesion between the particles of stone is insufficient to react against the propulsive force exerted by the driving wheels, hence the stones are loosened, and although the rubber tires with which the motor vehicles are equipped produce little dust by attrition or wearing away, the vehicle moving at high speed creates a partial vacuum. The current of air which then rushes in to re-establish the equilibrium picks up the small particles of stone displaced and loosened by the thrust of the driving wheels and distributes them in the form of dust, which is very disagreeable to other users

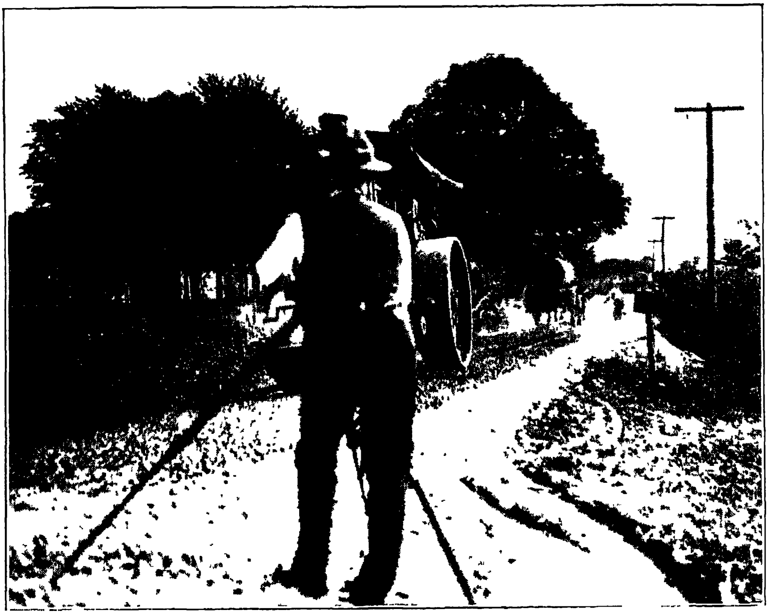


Fig 64 Rolling and Sprinkling Second Course of Macadam Road to Complete Binding Process

Courtesy of United States Department of Agriculture are thrown about and ground upon one another and thus increase the amount of fine material ready to be scattered as dust.

The frequent repetition of these actions causes the pavement to pit and disintegrate. The destructive effect is intensified, the greater the speed, and where the irregularities of the surfaces are such as to cause the wheels to leave it, there is produced a bounding motion that is continued for some distance and is particularly disastrous. Shearing of the road fabric is very severe on steep grades and curves due to the slipping of the driving wheels when the propulsive force is greater than the adhesion between the tire and the road surface. The damage arising from this is more extensive during wet weather and is intensified when the wheels are equipped with bars, chains, studs, and other anti-skidding devices.

The formation of dust and mud cannot be prevented absolutely, because all materials, by attrition and the disintegrating action of the elements, yield dust when dry and mud when wet. If the surface of a water-bound macadam road could be maintained in a moist condition, there would be no dust, but moistening with water even in cities, towns, and villages is expensive, and in rural districts the cost is prohibitive and the practice would be impossible, owing to the absence of water available for the purpose. Hence in dealing with existing road surfaces a remedy has been sought in more frequent cleansing and in the use of some substitute for water which would be cheap, effective, lasting, and easily applied. To meet this demand several "dust-laying" compositions have been placed on the market, and experiments have been made with some of these, but it has been demonstrated clearly that, with but few exceptions, they have a very temporary effect, and their application must be frequent and thorough.

Under the head of exceptions, that is, of the more or less permanent methods, are included the following: (1) the cementing of the surface stone by a bituminous cement or binder. When the binder is applied by the penetration method, the surface is described by the general term "bituminous-macadam"; and when it is desired to indicate the kind of binder, the descriptive names, "asphalt-macadam", "tar-macadam", etc., are used. When the binder is applied by the mixing method, the construction is called "bituminous-concrete". or specifically designated by the trade or patented name as, "bitulithic", "warrenite", "amiesite", "filbertine", "rock-asphalt", etc.; (2) binding the stone with hydraulic cement, the surface so formed being called "concrete-macadam", or "concrete pavement". These will be discussed later under their respective headings.

Turning to the details of the various temporary methods, we find the following:

(1) *Fresh Water.* This is the simplest remedy, but not always the most practicable nor the cheapest.

(2) *Sea Water.* This is a simple remedy but available only on the seacoast. The salts contained in sea water are highly antiseptic and deliquescent; a light sprinkling will suppress the dust for several hours. Its use, however, is objected to for the reason that it injures the varnish and running gear of vehicles, corrodes cast-iron street fittings, and when the road surface on which it has been used has dried the dust then produced, containing salt, injures food and other goods exposed to it. Moreover, after a few weeks' use the dust is converted into a pasty mud that adheres to the wheels and causes the surface of the road to be "picked up".

(3) *Deliquescent Salts.* The chief advantage of these salts is that their effect is more lasting than that of water. The salt used most extensively is calcium chloride obtained as a by-product in the manufacture of soda by the ammonia process. The salt may be applied either in solution or in the dry form. It takes up water rapidly and proves very efficient where the atmospheric moisture is sufficient to feed the salt. Glutrin, the commercial name for the waste sulphite liquor obtained in the manufacture of paper from wood pulp by the sulphite process, reduces the formation of dust, but the treatment must be repeated frequently. Waste molasses or "black strap" from sugar refineries mixed with milk of lime possesses good dust-suppressing qualities.

(4) *Coal-Tar Coating.* Refined coal tar applied either hot or cold in the form of a spray minimizes the production of dust, renders the surface waterproof, and reduces wear. The success attending its use depends upon the quality of the tar, the state of the weather, which must be clear and dry, the condition of the road surface, which must be dry and free from dust and dirt, and, in the case of

(5) *Solutions of Coal Tar and Petroleum.* Several patented preparations of coal tar are on the market. The principle of all is practically the same, namely, the solution of the tar or oil in water by a volatile agent, which on evaporation leaves a more or less insoluble coating on the road surface. The more favorably known of these preparations are "tarvia" and "westrumite".

(6) *Crude Petroleum and Residuum Oil.* Crude petroleum containing a large percentage of asphalt gives the best results. Petroleum having paraffin and naphtha as a base refuses to bind, and produces a greasy slime. The residuum oils obtained in the distillation of petroleum having asphaltum for a base have yielded good results in many cases.

Two methods are followed in applying the oil: (a) The surface of the road to be oiled is prepared by removing the dust with hand or power brooms. The oil, in the cold method, is applied by specially designed sprinkling wagons, at the rate of from one-third to one-half gallon per square yard. After being applied the oil is

or stone screenings and may or may not be

oil is applied once or twice a year according to whether

c is light or heavy. The surface of the road must be dry

e oil is applied.

(b) The oil is sprinkled over the surface and mixed with the dust. If the oil is merely sprinkled, the mixture of dust and oil made by the action of the traffic will become very sticky and will be removed in spots by adhering to the wheels. For the purpose of facilitating the handling and of securing a deeper penetration than is possible with cold oil, the oil is heated to a temperature of about 140° Fahrenheit and applied in the same manner as the cold oil.

(7) *Oil Tar and Creosote.* Oil tar is the residual liquid from the manufacture of carbureted water gas and oil gas. The tar used for road purposes is obtained by distilling the original tarry liquid to remove the light oil, naphthalene, and creosote. Various grades of tar are produced according to the temperature at which the distillation is stopped. The higher the temperature of distillation, the harder and more brittle the tar.

The oil tar either alone or mixed with creosote is applied in the same manner as coal tar.

Bituminous-Macadam

Features of Bituminous-Macadam. A bituminous-macadam wearing surface differs from the previously described water-bound broken-stone surface only in the kind of binder and the quality of the stone. The bituminous binder is prepared from asphalt, asphaltic oils, refined water-gas tars, refined coal tars, and combinations of refined tars and asphalts.

The bituminous binders adhere to comparatively porous and relatively soft stone, such as limestone, better than to the hard stones, such as trap and granite. Consequently, the stone used with the bituminous binders may be inferior in hardness and binding quality to that required for water-bound macadam.

Methods of Construction. The essentials necessary to the successful construction of a bituminous covering are: (1) the exclusion of both subsoil and surface water from the foundation; (2) a solid unyielding foundation; (3) a stone of suitable quality and size; (4) that the stone shall be entirely free from dust, otherwise the dust will interpose a thin film between the stone and the bituminous binder and prevent the latter from adhering to the stone; (5) if the stone is to be used hot, that it shall not be overheated; and if is to be used cold, that it shall be dry, for if wet or damp, the bituminous material will not adhere to it; (6) that the bituminous cement shall be of suitable quality; free from water, for which the stone has a greater affinity than for bitumen, and would thus prevent adhesion; free from ammoniacal liquor, which is apt to saponify some of the oily constitutents and thus render them capable of combining with water and therefore apt to be washed out; free from an excess of light oils and naphtha, which act as diluents and volatilize on the surface of the road, forming a skin that is not durable; free from an excess of free carbon, because it has no binding value and is liable to be converted into dust and mud.

Two general methods with various modifications in the minor details are employed for applying the bituminous binder to form the wearing surface, viz, the penetration method, and the mixing method.

Penetration Method. In this method, the stone is spread and packed slightly by rolling. The bituminous binder is then applied nozzle leading from a tank cart; or by a mechanical distributor u air pressure to discharge the material through nozzles that sp it in a finely divided stream or spray, Fig. 65. The binder is he usually by steam from the roller, but when hand pots are use is heated in kettles over fires. The quantity applied is abou gallons per square yard. After the binder is distributed, it is cov with a light coating of s dust, sand, or gravel, an rolling is continued. In cases, after the rolling is pleted, another applicati the binder is made at the of about one-half gallon square yard; this is call "paint coat" and is cov with a light sprinkling of s screenings.

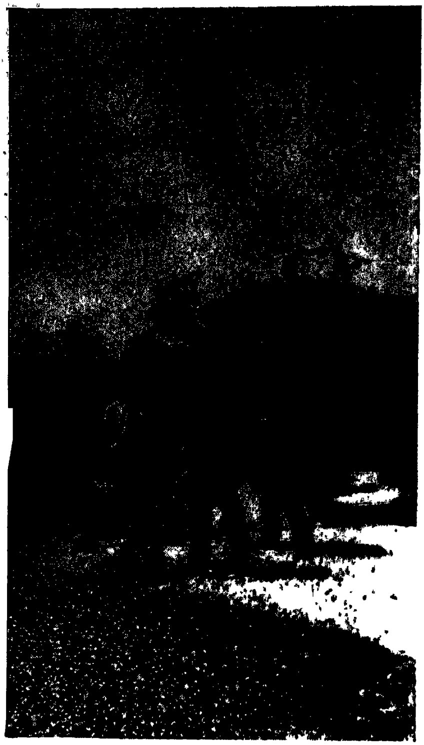


Fig. 65. Spreading Bituminous Binder by Pressure Nozzle, Penetration Method

Courtesy of Barrett Manufacturing Company, New York City

Mixing Method. In method the stone to be for the wearing surface, v ing in size from ½ to inches, is cleaned and d then mixed with a suffi quantity of the binder to all the stones thoroug The mixing is performe manual labor on a m board, Fig. 66, or by ra the stones through a ba liquid binder, or by pa through a mechanical mi machine, Fig. 67. The coated stones are spread upon the found in a layer having a thickness of about 3 inches and are cov with a light coating of stone screenings free from dust; then compacted by rolling, Fig. 68. Wherever the binder flushe the surface it is covered with screenings and rolled. When rolling is completed, the surplus screenings are swept from surface. The cleaned surface then is covered with a coat of

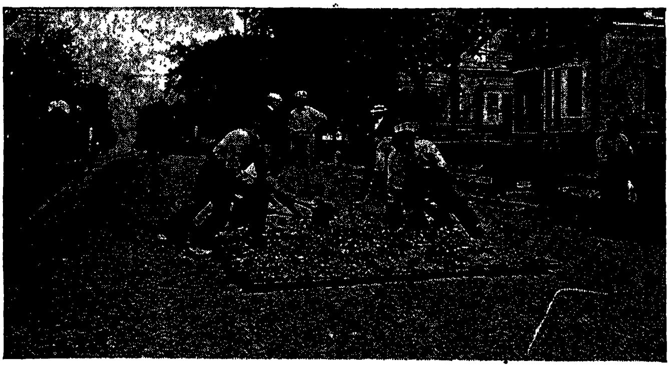


Fig 66. Hand Method of Mixing Stone and Binder
Courtesy of Barrett Manufacturing Company, New York City

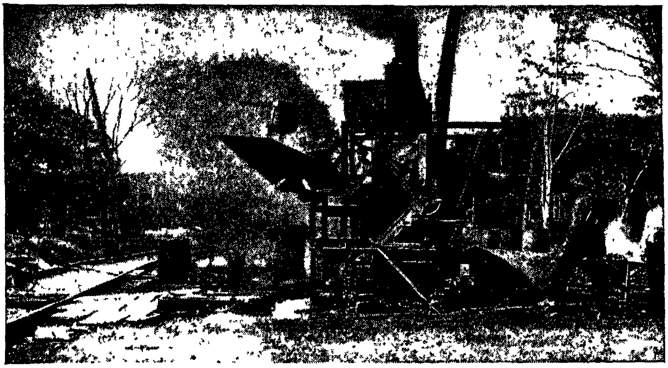
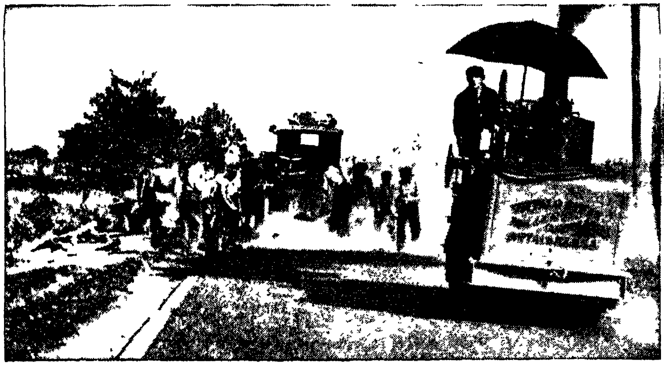


Fig 67 Machine Method of Mixing Stone and Binder
Courtesy of Barrett Manufacturing Company, New York City



binder called a "seal coat", for the purpose of insuring the wate proofing and complete filling of the voids, Fig. 69. For this coa about one-half gallon of binder is used per square yard of surfac Screenings again are spread and may or may not be rolled.

Advantages and Disadvantages of the Penetration Method. Th advantage of the penetration method is the ease and rapidity wit which it can be carried out, and the low cost for equipment and labo

The disadvantages of the penetration method are: (1) th difficulty of obtaining an absolutely uniform distribution of the binde thus producing "lean" and "fat" spots that will prove defective unde traffic; (2) it is wasteful, in that it is necessary to use more binde than actually is required to coat the stones and bind them togethe (3) it is difficult and sometimes impossible to use a binder of suff cient original consistency to produce a satisfactory bond, owing t the bitumen setting too rapidly when applied to cold stone.

Fig. 69. Spraying Seal Coat by Auto Truck, One-Half Gallon to the Yard
Courtesy of Barrett Manufacturing Company, New York City

Advantages and Disadvantages of the Mixing Method. Th advantages of the mixing method are: (1) the producing of a unifor fabric in which the cement is distributed uniformly and cemen each individual stone; (2) that construction can be carried on i colder weather than is permissible with the penetration metho If hot stone is used, a bitumen can be employed of such origin consistency as is required to sustain the traffic satisfactorily.

The disadvantage of the mixing method is the greater cos

TYPICAL ASPHALT PAVEMENT, SHOWING ALSO CONCRETE FOUNDATION IN FOREGROUND

due (1) to the increased labor, and (2) to the more elaborate equipment and apparatus required.

Bitulithic. Bitulithic is composed of stone, ranging in size from 2 inches to $\frac{1}{200}$ of an inch, and dust, which are dried, heated, and mixed in predetermined proportions, so as to reduce the voids to about 10 per cent, and cemented by a hot bituminous cement manufactured from either coal tar, asphalt, or a combination of both. The cement is added in sufficient quantity not only to coat every particle and to fill all of the remaining voids but with enough surplus to result in a rubbery and slightly flexible condition of the mixture after compression.

The mixture is spread, while hot, to such depth as will give a thickness of 2 inches after compressing with a 10-ton roller. After rolling, a composition coating called a "flush coat" is spread over the surface; this being covered while sticky with hot stone chips which are rolled until cool. The purpose of the stone chips is to form a gritty surface to prevent slipping.

Amiesite. Amiesite is a patented preparation of crushed stone or gravel, coated with an asphaltic cement. It is laid in two courses and a surface finish. The first course, composed of stone ranging from ½ inch to 1½ inches, is spread to a depth of 3 inches, blocks or strips of wood being used to insure uniformity of depth, then rolled once. The second course is composed of stone ½ inch and less, spread 1 inch deep, then rolled. The surface finish consists of screenings or sand, used in sufficient quantity to fill the voids.

Rock Asphalt. The rock asphalt most used in the United States is a sandstone containing from 7 to 10 per cent of asphalt. It is prepared for use by pulverizing and is used either hot or cold. It is spread upon the surface of the stone to a depth of about 1½ inches and rolled with a steam roller; the rolling is repeated daily for several days, or until the asphalt becomes hard.

Definitions of Bituminous Materials. The most recently adopted definitions of the bituminous materials employed in road construction are:

Native Bitumen. Native bitumen is a mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, which may be gases, liquids, viscous liquids, or solids and which are soluble

Artificial Bitumen. Artificial bitumen is produced by the destructive distillation of pyrobitumens and other substances of an organic nature; the bitumens so produced are commonly known as tars, the word tar being compounded with the name of the material which has been subjected to the process of destructive distillation, thus designating its origin, as, coal tar, oil tar, etc.

Bituminous. Bituminous refers to that which contains bitumen or constitutes a source of bitumen.

Emulsions. Emulsions are oily substances made mixable with water through the action of a saponifying agent or soap.

Fixed Carbon. Fixed carbon is the organic matter of the residual coke obtained upon burning hydrocarbon products in a covered vessel in the absence of free oxygen.

Fluxes. Fluxes are fluid oils and tars which are incorporated with asphalt and semi-solid or solid oil and tar residuums for the purpose of reducing or softening their consistency.

Residuums, Residual Petroleum, or Residual Oils. These are heavy viscous residues produced by the evaporation or distillation of crude petroleums until at least all of the burning oils have been removed.

Bituminous Cement. The bituminous cements or binders are prepared from (1) coal-, oil-, and water-gas tars; (2) asphaltic petroleums; (3) asphalt; and (4) combinations of asphalt and the residues of distillation from asphaltic petroleums.

Coal-Tar Binder. Coal-tar binder is the residue obtained by the distillation of the crude tar produced in the manufacture of illuminating gas and the manufacture of coke for metallurgical purposes; the required consistency is obtained by removing part or all of the contained oils. Owing to the difference in the temperatures employed in the two producing processes, the constituents of the tar, while identical in their characteristics, differ in their amount; the most marked difference being in the free carbon content, of which coke-oven tar has the least.

Oil-Gas Tar. Oil-gas tar is produced in the manufacture of gas from oil. The tarry residue is rather varied in character and is prepared for use by distillation; it usually contains a large amount of free carbon.

Water-Gas Tar. W

ture of carbureted water gas for illuminating purposes and results from the petroleum product employed for carbureting. It is a thin oily liquid containing a large percentage of water. It is prepared for road use by mechanical dehydration and distillation. It has a strong gassy odor when applied to the road, but this disappears in a few days.

Asphaltic Petroleums. Asphaltic petroleums are native petroleums which yield asphalts upon reduction. They are used in the crude state, or after the illuminating and other oil constituents have been removed by cracking or blowing.

Asphalt. Asphalt is the name by which the native semi-solid and solid bitumen is known. Asphalt is the most permanent type of binder and has been used for many years in the construction of sheet-asphalt pavements. The semi-solid or tarry varieties are called "malthas" and are the ones generally employed as a binder for broken-stone roads; the solid variety is used almost exclusively for street pavements. Rock asphalt or bituminous rock is the name given to a great variety of sandstone rocks more or less saturated or cemented by maltha or hard asphalt. In referring to rock asphalts it is customary to add the name of the locality where they are found, as "Kentucky rock asphalt", "Italian rock asphalt", etc. The semi-solid varieties are used in the natural state or after the water and volatile hydrocarbons have been removed by heating. The hard varieties are prepared by softening with a suitable flux.

Combinations of Asphalt and Distillation Residues. Combinations of asphalt and the residue from the distillation of tars and petroleums are made by adding either a refined maltha or a pulverized solid asphalt; the mixing being accomplished by the injection of compressed air through suitably formed nozzles.

Tests for Bituminous Materials. The bituminous materials are subjected to certain tests for the purpose of ascertaining their chemical and physical properties. The results of the tests are used in specifications to secure the furnishing of the desired quality of material and to control the processes of manufacture; also to form a record by which the behavior of the materials under similar and dissimilar service conditions can be compared. The complex character of the materials requires a suitably equipped laboratory

The tests are determinations of (1) amount of water contained; (2) materials soluble in water; (3) homogeneity at a temperature of 77° Fahrenheit; (4) specific gravity; (5) consistency or viscosity measured by a standard penetration machine; (6) ductility, or the distance the material can be drawn out before breaking; (7) toughness, or resistance to fracture under blows in an impact machine; (8) melting or softening point, measured by a thermometer while the temperature is raised through a water or oil bath; (9) distillation—the products yielded at different temperatures during continuous distillation in a suitable flask or retort are caught and weighed; (10) amount of free carbon—a sample is dissolved in carbon bisulphide, the solution filtered, and the insoluble residue weighed; (11) amount of fixed carbon—a sample is placed in a platinum crucible and heated until the emission of flame and smoke ceases, then is allowed to cool and the residue is weighed; and the difference between the weight of the sample and the residue is the fixed carbon; (12) paraffine—the presence of paraffine is determined by treating a sample with absolute ether, freezing the mixture, filtering the precipitate, evaporating and weighing the residue; (13) amount of bitumen contained—found by weighing a sample of the dried material, by adding carbon bisulphide to dissolve the bitumen, and by drying and weighing the residue after the extraction is completed. The loss is the amount of bitumen soluble in carbon bisulphide. A sample also is treated with naphtha and the character of the residue is noted as to whether it is sticky or oily.

Concrete Pavements

Construction Methods. Several methods with many variations are employed for the construction of concrete pavements The principal ones are: (1) grouting method, the construction being commonly called "concrete macadam"; (2) mixing method; (3) block or cube method.

Grouting Method. In this method the stone is spread upon the foundation and lightly rolled, after which a mixture of one part of cement and three parts of sand in the dry state is spread over the stone and swept into the interstices by brooms, then sprinkled with water and rolled; more cement and sand are spread, sprinkled, and rolled: the operation is repeated until the interstices are filled

A variation of this method, known as the Hassam paving, is made by spreading the stone, ranging in size from 1¼ inches to 2½ inches, and rolling it to a thickness of 4 inches, then filling the interstices with a grout composed of one part cement and three parts of sand mixed with water in a mixing machine, from which it flows over the surface, the machines being drawn along the roadway for this purpose; rolling is proceeded with at the same time and sufficient grout is applied to fill the interstices. On the foundation so prepared a wearing surface is formed; the stone is spread in the same manner as in the first course; the grout, composed of one part cement and two parts sand, mixed with sufficient water to make it very fluid, is applied by flowing over the surface of the compacted stone. The surface is finished by applying a thick grout composed of one part each cement, sand, and pea-sized trap rock.

Mixing Method. In this method the ingredients are combined into concrete by either hand or machine mixing; the concrete is deposited in place in one or two courses, the former being called "one-course" pavement and the latter "two-course" pavement. In the one-course method, the concrete mixed in the proportions of one part cement, one and one-half parts sand, and three parts stone is spread upon the prepared natural soil foundation and tamped to a thickness of about 6 inches.

In the two-course work the concrete mixed in the proportions of one part cement, two and one-half parts sand, and five parts stone is spread upon the prepared natural-soil foundation and compacted by rolling or tamping to the required thickness. On its surface, and before the cement has set, the wearing surface of about 2 inches in thickness is placed and tamped to the required contour. The mixtures used for the wearing surface vary, being composed of sand and cement, or of sand, cement, and small broken stone. The wearing surface of the Blome pavement is composed of one part cement and one and one-half parts of aggregate, which is made up of 50 per cent ¼-inch, 30 per cent ⅛-inch, and 20 per cent $\frac{1}{10}$-inch granite screenings. The surface is formed into 4½-inch by 9-inch blocks by cutting grooves ½ inch wide and ¼ inch deep by means of special tools.

Materials. The materials used in the construction of concrete pavements should be selected with care. The stone should be a hard tough rock, free from dust and dirt, and graded so as to reduce voids to the minimum. The sand should be free from loam, clay, vegetable and organic matter, and should grade from coarse to fine. The cement should be of a quality to meet the standard tests. The water should be clean and free from organic matter, alkalies, and acids. Rapid drying of the concrete should be prevented by covering it with a canvas which is kept moistened with water for several hours; after its removal the surface should be covered with sand or earth which is to be kept moist for a period of two weeks. Improperly mixed or constructed concrete pavement will wear unevenly, crack, and rapidly become very defective.

Expansion Joints. To provide for the expansion and contraction of the concrete under changes of temperature, expansion joints are formed at intervals ranging from 15 to 50 feet. The edges of the joints are protected from injury by angle irons, and the space between them, about ½ inch, is filled with a bituminous cement which extends the full depth of the concrete. When the concrete is laid between curbs longitudinal joints from ½ inch to 1½ inches wide, filled with bituminous cement, are formed along the curb.

Reinforced-Concrete Pavement. Concrete pavements reinforced with steel in the form of woven-wire, Fig. 70, expanded metal, and round bars are constructed in two courses, the reinforcement being placed between the foundation course and the wearing surface

Concrete with Bituminous Surface. In this type the surface of the concrete pavement, constructed by either the grouting or mixing method, is covered with a bituminous cement made from either asphalt, coal tar, or a mixture of both.

Block or Cube Pavement. In this type of pavement, blocks or cubes of concrete are molded in a machine or cast in molds. The blocks are stacked and allowed to season for three months, during which time they are wet twice a day. They are laid by hand on a sand cushion spread upon the foundation, then are brought to a firm bearing and uniform surface by rolling with a light roller. The surface is covered with a layer of sand or sandy loam which is broomed and flushed by water into the joints and the rolling is repeated; after which the surface is covered with a layer of sand, and the traffic then admitted.

A variation from the methods described is made in the patented pavement "rocmac". This is composed of broken stone cemented by silicate of lime, obtained by mixing powdered carbonate of lime with a solution of silicate of soda and sugar. The silicate of lime mortar is spread upon the foundation to a depth of about 2 inches, over which the broken stone is distributed to such a depth as will give, when compacted, a depth of about 4 inches. It is rolled and sprinkled with water until the mortar flushes to the surface, and then is covered with a layer of stone screenings and finally opened to traffic.

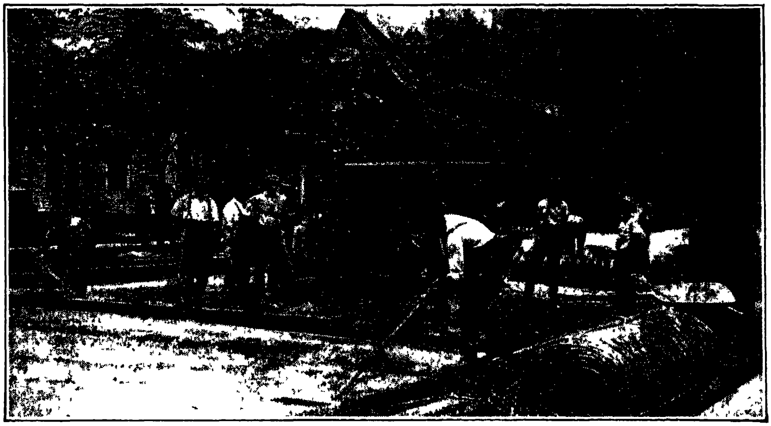


Fig. 70. Laying Reinforced Concrete Road Woven Wire Fabric in Foreground Ready to Be Placed between Upper and Lower Coat
Courtesy of Municipal Engineering and Contracting Company, Chicago

MAINTENANCE AND IMPROVEMENT OF ROADS

Repair and Maintenance of Broken=Stone Roads. These terms frequently but erroneously are used interchangeably. Repair means the restoring of a surface so badly worn that it cannot be maintained in good condition. A well-maintained road should not require repairs for a considerable length of time. The maintenance of a road is the keeping of it, as nearly as practicable, in the same condition as it was when constructed.

Good maintenance comprises: (1) constant daily attention to repair the ravages of traffic and the elements; (2) cleansing to remove the detritus caused by wear, the horse droppings, and other refuse; and (3) application of water or other dust layer.

When the surface of a water-bound broken-stone road requires to be renewed, it is loosened and broken up by scarifying, the new stone spread, rolled, watered, and bound in the same manner as in new construction; or the old surface is cleansed from dust and other matter by sweeping and washing, and the new stone spread upon it, compacted and finished as in new construction.

The resurfacing of water-bound roads with a bituminous construction is becoming common. The methods employed are the same as heretofore described under "bituminous macadam".

Systems of Maintenance. Several systems for maintaining roads are in use, the one yielding the best results being that which provides for the continuous employment of skilled workmen. The men so employed become familiar with the peculiarities of the sections in their charge and with the best way to deal with them. Efficient maintenance requires that the surfaces be kept smooth so that surface water may flow away rapidly and that the injury caused by traffic on uneven surfaces may be avoided; that incipient ruts, hollows, and depressions be eliminated by cutting out the area involved in the form of a square or rectangle and filling with new material; that dust and horse droppings be removed; that loose stones be removed; that gutters be clear so the rain water may be removed quickly; that ditches and culverts be cleaned out in advance of the spring and fall rains; that weeds and grass be removed from the earth shoulders, and that these and the dust sweepings be not left on the sides of the road to be redistributed, but be removed immediately and disposed of in such manner as will not cause injury; that bridges be examined and repaired at least twice a year.

Improvement of Existing Roads. The improvement of existing roads may be divided into three branches: (1) rectification of alignment; (2) drainage; (3) improvement of the surface.

The first of these consists in the application of the principles which have been laid down for the location, etc., of new roads and will include straightening the course by eliminating unnecessary curves and bends; improving the grade either by avoiding or cutting down hills and by embanking valleys; increasing the width where necessary, and rendering it uniform throughout.

The second, or drainage, consists in applying the principles laid down for the drainage of new roads, and in constructing the works necessary to give them effect.

The third, or improvement of the surface, consists in improving the surface by any of the methods previously described and that the funds available will permit.

Value of Improvement. The improvement of roads is chiefly an economical question relating to the waste of effort and to the saving of expenditure. Good roads reduce the resistance to locomotion, and this means reduction of the effort required to move a given load. Any effort costs something, and so the smallest effort costs the least, and therefore the smoothest road saves the most money for everyone who traverses it with a vehicle.

Cost of Improvement. Before undertaking any improvement generally it is required to know the cost of the proposed improvement and the benefits it will produce. In the improvement of roads the amount of money that may be expended profitably for any proposed improvement may be calculated with sufficient accuracy by obtaining first the following data: (1) the quantity and quality of the traffic using the road; (2) the cost of haulage; (3) plan and profile of the road; and (4) character and cost of the proposed improvements. From the data ascertain the total annual traffic and the total annual cost of hauling it. Next, calculate the annual cost of hauling the given tonnage over the road when improved. Then the difference between the two costs will represent the annual interest on the sum that may be expended in making the improvement. For example, if the annual cost of haulage over the existing road is $10,000 and the cost for hauling the same tonnage over the improved road will be $7000, the difference, $3000, with money at 6 per cent per annum, represents the sum of $50,000 that logically may be appropriated to carry out the improvement.

Traffic Census. The direction, character, and amount of traffic using a road is obtained by direct observation during different seasons of the year. As a preliminary to observing the traffic it is usual to determine the weight of the vehicles; this is done by weighing typical vehicles and by establishing an average weight for each type. The traffic is classified according to the motive power—as horse-drawn vehicles and motor vehicles. Each of these classes is

TABLE XI

Traffic Census

Average Hours per Day..................; for............Days
Taken at
By

			Empty Vehicles		Loaded Vehicles	
			Nov to March	Aug. to Oct.	Nov. to March	Aug. to Oct.
Horse	Pleasure					
	Commercial	Light				
		Medium				
		Heavy				
Motor	Pleasure	Motorcycles				
		Runabouts				
		Touring cars				
	ommercial	Light				
		Medium				
		Heavy				

divided into pleasure and commercial traffic, the latter class being subdivided into loaded and non-loaded vehicles. The number of horses to a vehicle in horse-drawn traffic and the speed of motor vehicles may be noted. A summary of data is suggested in Table XI.

The observations are made from 6 a.m. to 6 p.m., during a period of seven days each month, with occasional observations, from 6 p.m. to 6 a.m. or for the entire 24 hours if the amount of traffic requires it.

The weight of the traffic is ascertained by multiplying the number of each kind of vehicle by the average weight established for that type.

LAYING TRINIDAD SHEET ASPHALT PAVING IN FIFTH AVENUE, NEW YORK CITY

HIGHWAY CONSTRUCTION

PART II

CITY STREETS AND HIGHWAYS

The first work requiring the skill of the engineer is the laying out of town sites properly, especially with reference to the future requirements of a large city, where any such possibility exists. Few if any of our large cities were so planned. The same principles, to a limited extent, are applicable to all towns or cities. The topography of the site should be studied carefully, and the street lines adapted to it. These lines should be laid out systematically, with a view to convenience and comfort, and also with reference to economy of construction, future sanitary improvements, grades, and drainage.

Arrangement of City Streets. Generally, the best method of laying out streets is in straight lines, with frequent and regular intersecting streets, especially for the business parts of a city. When there is some centrally located structure, such as a courthouse, city hall, market, or other prominent building, it is very desirable to have several diagonal streets leading thereto. In the residence portions of cities, especially if on hilly ground, curves may replace straight lines with advantage, by affording better grades at less cost of grading, and by improving property through avoiding heavy embankments or cuttings.

Width of Streets. The width of streets should be proportioned to the character of the traffic that will use them. No rule can be laid down by which to determine the best width of streets; but it may be said safely that a street which is likely to become a commercial thoroughfare should have a width of not less than 120 feet between the building lines—the carriage-way 80 feet wide, and the sidewalks each 20 feet wide.

In streets occupied entirely by residences a carriage-way 32 feet as great as desired. The sidewalks may be any amount over 10 feet which fancy dictates. Whatever width is adopted for them, not more of it than 8 feet need be paved, the remainder being occupied with grass and trees.

Street Grades. The grades of city streets depend upon the topography of the site. The necessity of avoiding deep cuttings or high embankments which seriously would affect the value of adjoining property for building purposes, often demands steeper grades than are permissible on country roads. Many cities have paved streets on 20 per cent grades. In establishing grades through unimproved property, they usually may be laid with reference to securing the most desirable percentage within a proper limit of cost. But when improvements already have been made and have been located with reference to the natural surface of the ground, the matter of giving a desirable grade without injury to adjoining property frequently is one of extreme difficulty. In such cases it becomes a question of how far individual interests shall be sacrificed to the general good. There are, however, certain conditions which it is important to bear in mind: (1) That the longitudinal crown level should be sustained uniformly from street to street intersection, whenever practicable. (2) That the grade should be sufficient to drain the surface. (3) That the crown levels at all intersections should be extended transversely, to avoid forming a depression at the junction.

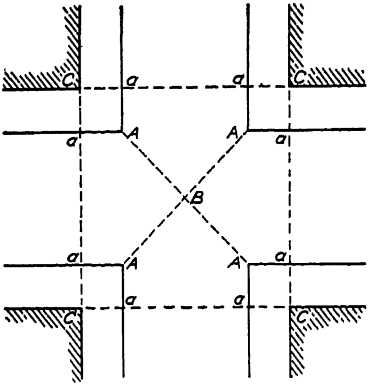


Fig. 71. Diagram Showing Arrangement of Grades at Street Intersections

Arrangements of Grades at Street Intersections. The best arrangement for intersections of streets when either or both have much inclination is a matter which requires careful consideration and upon which much diversity of opinion exists. No hard or fast rule can be laid down; each will require special adjustment. The best and simplest method is to make level the rectangular space *aaaaaaaa*, placing gulleys at *AAAA* and the catch basins at *ccc*. When this method is not practicable, adopt such a grade (but one not exceeding 2½ per cent) that the rectangle *AAAA* shall appear to be nearly level; but to secure this it must have actually a considerable dip in the direction of the slope of the street. If steep grades are continued across intersections, they introduce side slopes in the streets thus crossed, which are troublesome, if not dangerous, to vehicles turning the corners, especially the upper ones. Such intersections are especially objectionable in rainy weather. The storm water will fall to the lowest point, concentrating a large quantity of water at two receiving basins, which, with a broken grade, could be divided among four or more basins.

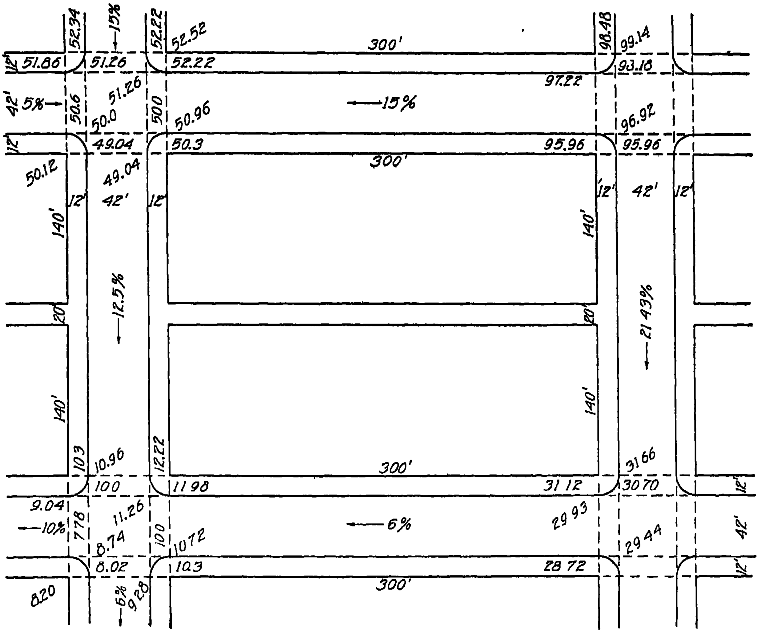


Fig 72 Diagram Showing [illegible] Intersections for Steep Grades in [illegible]

Fig. 72 shows the arrangement of intersections in steep grades adapted for the streets of Duluth, Minnesota. From this it will be seen that at these intersections the grades are flattened to 3 per cent for the width of the roadway of the intersecting streets, and that intersecting sidewalks. Grades of less amount on roadway or sidewalk are continuous. The elevation of block corners is found by adding together the curb elevations at the faces of the block corners, and 2½ per cent of the sum of the widths of the two sidewalks at the corner, and dividing the whole by two. This gives an elevation equal to the average elevation of the curbs at the corners, plus an average rise of 2½ per cent across the width of the sidewalk.

"Accommodation summits" have to be introduced between street intersections in two general cases: (1) in hilly localities, to avoid excessive excavation; and (2) when the intersecting streets are level or nearly so, for the purpose of obtaining the fall necessary for surface drainage.

The elevation and location of such a summit may be calculated as follows: Let *A*, Fig. 73, be the elevation of the highest corner; *B*, the elevation of the lowest corner; *D*, the distance from corner to corner; and *R*, the rate of the accommodation grade. The elevation of the summit is equal to

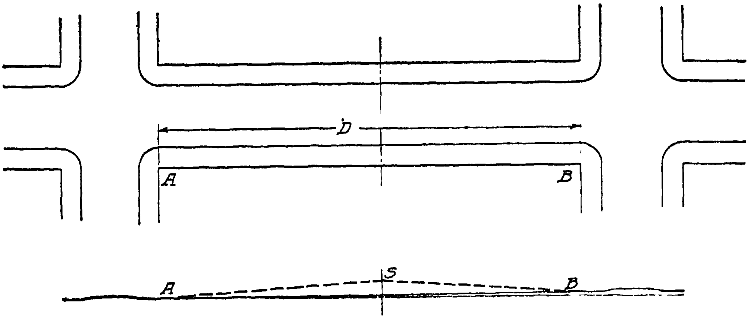
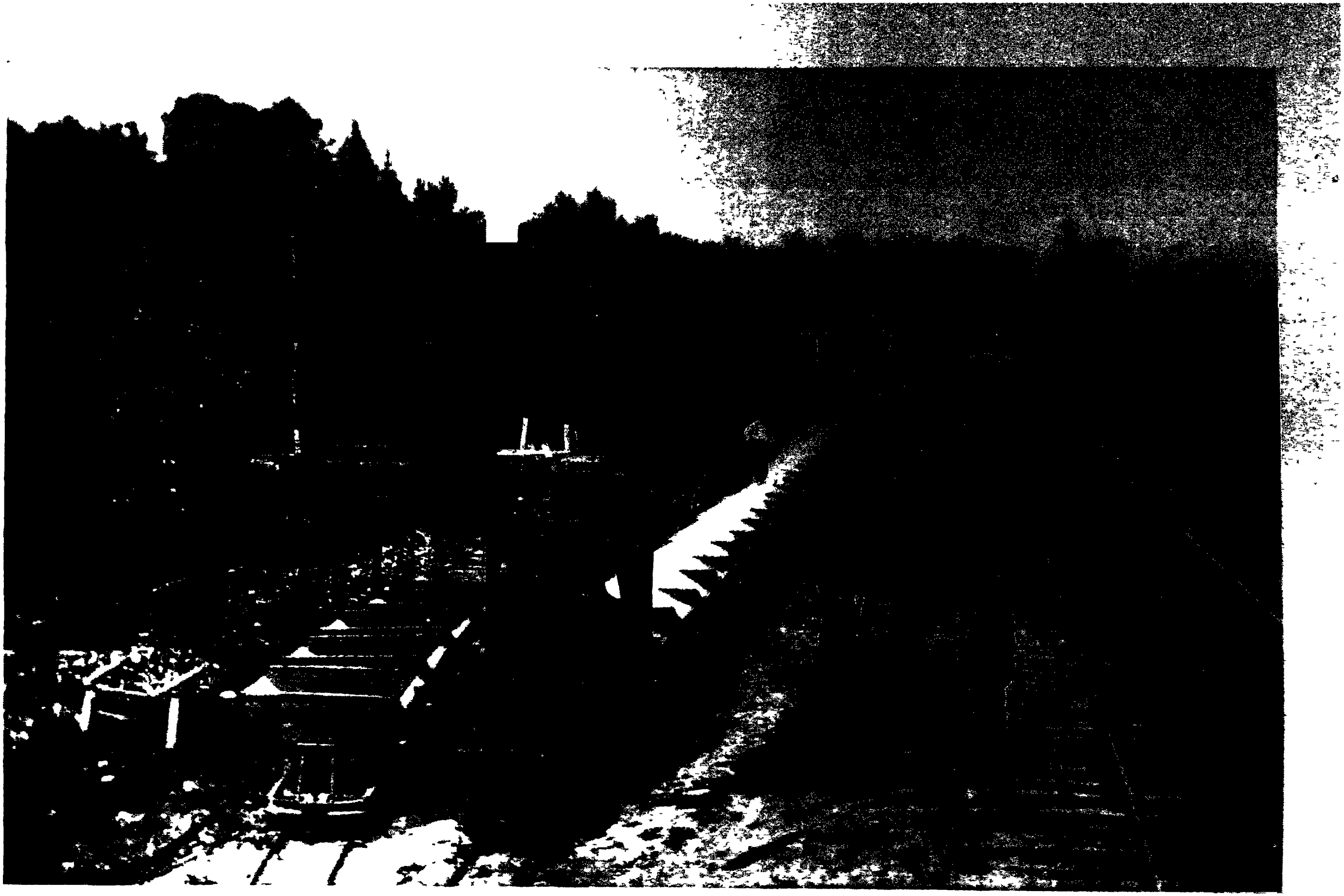


Fig. 73. Diagrams for Calculating "Accommodation Summits" between Street Intersections

$$\frac{D\times R+A+B}{2}$$

The distance from *A* or *B* is found by subtracting the elevation of either *A* or *B* from this quotient, and dividing the result by the rate of grade. Or the summit may be located mechanically by specially prepared scales. Prepare two scales divided to correspond to the rate of grade; that is, if the rate of grade be 1 foot per 100 feet, then one division of the scale should equal 100 feet on the map scale.



CRUSHED ROCK TRAIN DRAWN BY KOPPEL "DINKY" ENGINE

These engines and dump cars are found extremely serviceable in laying concrete highways. Roads thus prepared are much superior to usual gravel turnpike and can be laid at surprisingly low cost.

Courtesy of Orenstein-Arthur-Koppel Company, Chicago

These divisions may be subdivided into tenths. One scale should read from right to left, and one from left to right.

To use the scales, place them on the map so that their figures correspond with the corner elevations; then, as the scales read in opposite directions, there is of course some point at which the opposite readings will be the same. This point is the location of the summit, and the figures read off the scale give its elevation. If the difference in elevation of the corners is such as not to require an intermediate summit for drainage, it will be apparent as soon as the scales are placed in position.

When an accommodation summit is employed, it should be formed by joining the two straight grade lines by a vertical curve, as described in Part I. The curve should be used both in the crown of the street and in the curb and footpath.

Where the grade is level between intersections, sufficient fall for surface drainage may be secured without the aid of accommodation summits, by arranging the grades as shown in Fig. 74. The curb is set level between the corners; a summit is formed in the gutter; and receiving basins are placed at each corner.

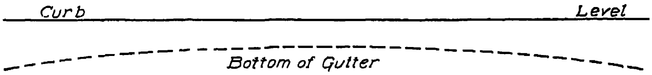


Fig 74 Diagram Showing Arrangement of Grades to Avoid "Accommodation Summits"

Transverse Grade. In its transverse grade the street should be level; that is, the curbs on opposite sides should be at the same level, and the street crown rise equally from each side to the center. But in hillside streets this condition cannot be fulfilled always, and opposite sides of the street may differ as much as 5 feet. In such cases the engineer will have to use his discretion as to whether he

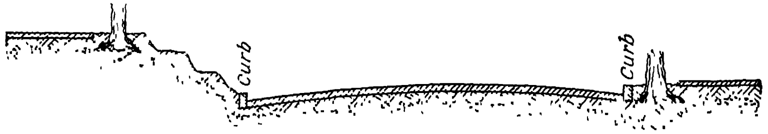


Fig 75 Street with Unequal Transverse Grade but with Level Street

the whole street by the lower gutter, or adopt the three-curb method and sod the slope of the higher side.

In the improvement of old streets with the sides at different levels, much difficulty will be met, especially where shade trees have to be spared. In such cases, recognized methods have to be abandoned, and the engineer will have to adopt methods of overcoming the difficulties in accordance with the conditions and necessities of each particular case. Figs. 75, 76, and 77 illustrate several typical arrangements in the cases of streets where the opposite sides are at different levels.

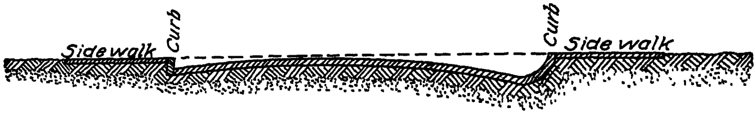


Fig. 76. Street with Unequal Transverse Grade Inclined so as to Drain by Lower Gutter

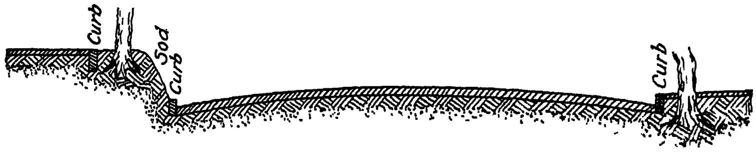


Fig 77. Street with Unequal Transverse Grade with Three Curbs and Higher Slope Sodded

Transverse Contour or Crown. The reason for crowning a pavement—i.e., making the center higher than the sides—is to provide for the rapid drainage of the surface. The most suitable form for the crown is the parabolic curve, which may be started at the curb line, or at the edge of the gutter adjoining the carriage-way, about one foot from the curb. Fig. 78 shows this form, which is obtained by dividing the abscissa, or width from the center of the street to the gutter, into ten equal parts, and by dropping perpendiculars at each of these divisions, the lengths of which are determined by multiplying the rise at the center by the square of the

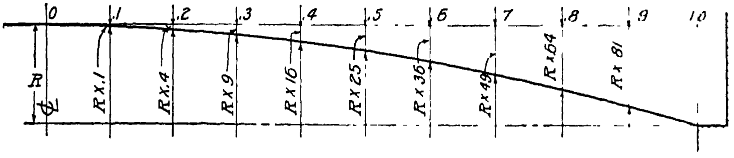


Fig 78 Method of Obtaining Transverse Contour or Crown of a Road

TABLE XII

Rise of Pavement Center above Gutter for Different Paving Materials

Paving Material	Proportions of Rise at Center to Width of Carriage-Way
Wood blocks	1 : 100
Stone blocks	1 : 80
Brick	1 : 80
Asphalt	1 : 80

successive values of the abscissas. The amounts thus obtained can be added to the rod readings; and the stakes, set at the proper distance across the street, with their tops at this level, will give the required curve.

The amount of transverse rise, or the height of the center above the gutters, varies with the different paving materials; smooth pavements requiring the least, and rough ones and earth the greatest rise. The rise is generally stated in a proportion of the width of the carriage-way. The most suitable proportions are shown in Table XII.

Drainage of Streets. *Sub-Foundation Drainage.* The sub-foundation drainage of streets cannot be effected by transverse drains, because of their liability to disturbance by the introduction of gas, water, and other pipes.

Longitudinal drains must be depended upon entirely; they may be constructed of the same materials and in the same manner as road drains. The number of these longitudinal drains must depend upon the character of the soil. If the soil is moderately retentive, a single row of tiles or a hollow invert placed under the sewer in the center of the street generally will be sufficient; or two rows of tiles may be employed, one placed outside each curb line. If, on the other hand, the soil is exceedingly wet and the street very wide, four or more lines may be employed. These drains may be permitted to discharge into the sewers of the transverse streets.

Surface Drainage. The removal of water falling on the street surface is provided for by collecting it in the gutters, from which it is discharged into the sewers or other channels by means of catch basins placed at all street intersections and dips in the street grades.

Gutters. The gutters must be of sufficient depth to retain all the path. The depth should never be less than 6 inches, and very rarely need be more than 10 inches.

Catch Basins. Catch basins are of various forms, usually circular or rectangular, built of brick masonry coated with a plaster of Portland cement. Whichever form is adopted, they should fulfill the following conditions:

(1) The inlet and outlet should have sufficient capacity to receive and discharge all water reaching the basin.

(2) The basins should have sufficient capacity below the outlet to retain all sand and road detritus, and prevent its being carried into the sewer.

(3) They should be trapped so as to prevent the escape of sewer gas. (This requirement frequently is omitted, to the detriment of the health of the people.)

(4) They should be constructed so that the pit can be cleaned out easily.

(5) The inlet should be constructed so as not to be choked easily by leaves or débris.

(6) They must offer the least possible obstruction to traffic.

(7) The pipe connecting the basin to the sewer should be freed easily of any obstruction.

The bottoms of the basins should be 6 or 8 feet below the street level; and the water level in them should be from 3 to 4 feet lower than the street surface, as a protection against freezing.

The capacity and number of basins will depend upon the area of the surface which they drain.

In streets having level or light longitudinal grades, gullies may be formed along the line of the gutter at such intervals as may be found necessary.

Catch basins usually are placed at the curb line. In several cities, the basin is placed in the center of the street, and connects to inlets placed at the curb line. This reduces the cost of construction and cleaning, and removes from the sidewalk the dirty operations of cleaning the basins.

Catch basins and gully pits require cleaning out at frequent intervals; otherwise the odor arising from the decomposing matter contained in them will be very offensive. No rule can be laid down for the intervals at which the cleaning should be done, but they must be cleaned often enough to prevent the matter in them from putrefying. There is no uniformity of practice observed by cities in this matter; in some, the cleaning is done but once a year; in others, after every rain-storm; in still others, at intervals of three or four months; while in a few cities the basins are cleaned out once a month.

FOUNDATIONS

The stability, permanence, and maintenance of any pavement depend upon its foundation. If the foundation is weak, the surface soon will settle unequally, forming depressions and ruts. With a good foundation, the condition of the surface will depend upon the material employed for the pavement and upon the manner of laying it.

The essentials necessary to the forming of a good foundation are:

(1) The entire removal of all vegetable, perishable, and yielding matter. It is of no use to lay good material on a bad substratum.

(2) The drainage of the subsoil wherever necessary. A permanent foundation can be secured only by keeping the subsoil dry; for, where water is allowed to pass into and through it, its weak spots will be discovered quickly, and settlement will take place.

(3) The thorough compacting of the natural soil by rolling with a roller of proper weight and shape until there is formed a uniform and unyielding surface.

(4) The placing on the natural soil so compacted of a thickness of an impervious and incompressible material sufficient to cut off all communication between the soil and the bottom of the pavement.

The character of the natural soil over which the roadway is to be built has an important bearing upon the kind of foundation and the manner of forming it; each class of soil will require its own special treatment. Whatever its character, it must be brought to a dry and tolerably hard condition by draining and rolling. Sand and gravels which do not hold water, present no difficulty in securing a solid and secure foundation; clays and soils retentive of water are the most difficult. Clay should be excavated to a depth of at least 8 inches below the bottom of the finished covering; and the space so excavated should be filled in with sand, furnace slag, ashes, coal dust, oyster shells, broken brick, or other materials which are not absorbent of and effectually improved by laying cross drains with open joints at intervals of 50 or 100 feet. These drains should be not less than 18 inches below the surface, and the trenches should be filled with gravel. They should be 4 inches in internal diameter, and should empty into longitudinal drains.

Sand and planks, gravel and broken stone successively have been used to form the foundation for pavements; but, although eminently useful materials, their application to this purpose always has been a failure. Being inherently weak and possessing no cohesion, the main reliance for both strength and wear must be placed upon the surface covering. This covering—usually (except in case of sheet asphalt) composed of small units, with joints between them varying from ½ inch to 1½ inches—posesses no elements of cohesion; and under the blows and vibrations of traffic the independent units or blocks will settle and be jarred loose. On account of their porous nature, the subsoil quickly becomes saturated with urine and surface waters, which percolate through the joints; winter frosts upheave them; and the surface of the street becomes blistered and broken up in dozens of places.

Concrete. As a foundation for all classes of pavement (broken stone excepted), hydraulic-cement concrete is superior to any other. When properly constituted and laid, it becomes a solid, coherent mass, capable of bearing great weight without crushing. If it fail at all, it must fail altogether. The concrete foundation is the most costly, but this is balanced by its permanence and by the saving in the cost of repairs to the pavement which it supports. It admits of access to subterranean pipes with less injury to the neighboring pavement than any other, for the concrete may be broken through at any point without unsettling the foundation for a considerable distance around it, as is the case with sand or other incoherent material; and when the concrete is replaced and set, the covering may be reset at its proper level, without the uncertain allowance for settlement which is necessary in other cases.

Thickness of Course. The thickness of the concrete bed must be proportioned by the engineer; it should be sufficient to provide against breaking under transverse strain caused by the settlement of the subsoil. On a well-drained soil, 6 inches will be found sufficient; but in moist and clayey soils, 12 inches will not be excessive. On such soils a layer of sand or gravel, spread and compacted before placing the concrete, will be found very beneficial.

The proportions of the ingredients required for the manufacture of concrete are ascertained by measuring the voids in each ingredient. The strongest concrete will be produced when the volume of cement is slightly in excess of that required to fill the voids in the sand, and the volume of the combined cement and sand exceeds by about 10 per cent the volume of the voids in the stone or other material used for the aggregate. Concrete frequently is mixed in the arbitrary proportions of 1 part of cement, 3 parts of sand, and 6 parts of stone, and although the results have been satisfactory, the proportions may not be the most economical.

The ingredients of the concrete should be thoroughly mixed with just sufficient water to produce a plastic mass, without any surplus water running from it. After mixing, the concrete should be deposited quickly in place, and brought to a uniform surface and thickness by raking, then tamped until the mortar flushes to the surface, then left undisturbed until set. The surface of concrete laid during dry, warm, weather should be protected from the drying action of the sun while the initial setting is in progress. This may be accomplished by sprinkling with water as frequently as the rate of evaporation demands or by covering it with a layer of damp sand, straw, hay, or canvas. During freezing weather it is customary to suspend the laying of concrete for the reason that alternate freezing and thawing disintegrate it.

Measuring Voids in the Stone and Sand. The simplest method for measuring the voids and one sufficiently accurate for the manufacture of concrete is the "pouring method" in which a suitable vessel of known capacity (usually one cubic foot) is filled with the material, in which it is desired to ascertain the voids. Water then is poured into the vessel until its surface is flush with the surface of the material. The water is measured, and its amount is considered to equal the total of the voids.

STONE-BLOCK PAVEMENTS

Stone blocks commonly are employed for pavements where traffic is heavy. The material of which the blocks are made should possess sufficient hardness to resist the abrasive action of traffic, and

TABLE XIII

Specific Gravity, Weight, Resistance to Crushing, and Absorptive Power of Stones

Material	Specific Gravity		Weight (lb. per cu. ft.)		Resistance to Crushing (lb per sq. in.)		Percentage of Water Absorbed	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Granite	2.60	2.80	163	176	12,000	35,000	0.066	0.155
Trap	2.86	3.03	178	189	19,000	24,000	0.000	0.019
Sandstone	2.23	2.75	137	170	5,000	18,000	0.410	5.480
Limestone	1.90	2.75	118	175	7,000	20,000	0.200	5.000
Brick, paving	1.95	2.55			10,000	20,000		

sufficient toughness to prevent them from being broken by the impact of loaded wheels. The hardest stones will not give necessarily the best results in the pavement, since a very hard stone usually wears smooth and becomes slippery. The edges of the block chip off, and the upper face becomes rounded, thus making the
it very rough.

stone sometimes is tested to determine its strength, resistance to abrasion, etc.; but, as the conditions of use are quite different from those under which it may be tested, such tests are seldom satisfactory. However, examination of a stone as to its structure, the closeness of its grain, its homogeneity, porosity, etc., may assist in forming an idea of its value for use in a pavement. A low degree of permeability usually indicates that the material will not be greatly affected by frost. For data see Table XIII.

Materials. *Granite.* Granite is employed more extensively for stone-block paving than is any other variety of stone; and because of this fact, the term "granite paving" is generally used as being synonymous with stone-block paving. The granite employed should be of a tough, homogeneous nature. The hard, quartz granites usually are brittle, and do not wear well under the blows of horses' feet or the impact of vehicles; granite containing a high percentage of feldspar will be injuriously affected by atmospheric changes; and granite in which mica predominates will wear rapidly on account of its laminated structure. Granite possesses the very important property of splitting in three planes at right angles to one another, so that paving blocks may readily be formed with nearly plane faces and square corners. This property is called the rift or cleavage.

Sandstones. Sandstones of a close-grained, compact nature often give very satisfactory results under heavy traffic. They are less hard than granite, and wear more rapidly, but do not become smooth and slippery. Sandstones are generally known in the market by the name of the quarry or place where produced as "Medina", "Berea", etc.

Trap Rock. Trap rock, while answering well the requirements as to durability and resistance to wear, is objectionable on account of its tendency to wear smooth and become slippery; it is also difficult to break into regular shapes.

Limestone. Limestone usually has not been successfully employed in the construction of block pavements, on account of its lack of durability against atmospheric influences. The action of frost commonly splits the blocks; and traffic shivers them, owing to the lamination being vertical.

Cobblestone Pavement. Cobblestones bedded in sand possess the merit of cheapness, and afford an excellent foothold for horses; but the roughness of such pavements requires the expenditure of a large amount of tractive energy to move a load over them. Aside from this, cobblestones are entirely wanting in the essential requisites of a good pavement. The stones being of irregular size, it is almost impossible to form a bond or to hold them in place. Under the action of the traffic and frost, the roadway soon becomes a mass of loose stones. Moreover, cobblestone pavements are difficult to keep clean, and very unpleasant to travel over.

Belgian=Block Pavement. Cobblestones were displaced by pavements formed of small cubical blocks of stone. This type of pavement was laid first in Brussels, thence imported to Paris, and from there taken to the United States, where it has been widely known as the "Belgian-block" pavement. It has been largely used in New York City, Brooklyn, and neighboring towns, the material being trap rock obtained from the Palisades on the Hudson River.

The stones, being of regular shape, remain in place better than cobblestones; but the cubical form (usually 5 inches in each dimension) is a mistake. The foothold is bad; the stones wear round; and the number of joints is so great that ruts and hollows are quickly formed. This pavement offers less resistance to traction than cobblestones, but it is almost equally rough and noisy.

Granite-Block Pavement. The Belgian block gradually has been displaced by the introduction of rectangular blocks of granite. Blocks of comparatively large dimensions were employed at first. They were from 6 to 8 inches in width on the surface, from 10 to 20 inches in length, with a depth of 9 inches. They merely were placed in rows on the subsoil, perfunctorily rammed, the joints filled with sand, and the street thrown open to traffic. The unequal settlement of the blocks, the insufficiency of the foothold, and the difficulty of cleansing the street, led to the gradual development of the latest type of stone-block pavement, which consists of narrow, rectangular blocks of granite, properly proportioned, laid on an unyielding and impervious foundation, with the joints between the blocks filled with an impermeable cement.

Experience has proved beyond doubt that this latter type of pavement is the most enduring and economical for roadways subjected to heavy and constant traffic. Its advantages are many, while its defects are few.

Advantages.

(1) Adapted to all grades.
(2) Suits all classes of traffic.
(3) Exceedingly durable.
(4) Foothold, fair.
(5) Requires but little repair.
(6) Yields but little dust or mud.
(7) Facility for cleansing, fair.

Defects.

(1) Under certain conditions of the atmosphere, the surface of the pavement becomes greasy and slippery.

(2) The incessant din and clatter occasioned by the movement of traffic is an intolerable nuisance; it is claimed by many physicians that the noise injuriously affects the nerves and health of persons who are obliged to live or do business in the vicinity of streets so paved.

(3) Horses constantly employed upon it soon suffer from the continual jarring produced in their legs and hoofs, and quickly wear out.

(4) The discomfort of persons riding over the pavement is very great, because of the continual jolting to which they are subjected.

(5) If stones of an unsuitable quality are used—for example, those that polish—the surface quickly becomes slippery and exceedingly unsafe for travel.

Blocks. *Size and Shape.* The proper size of blocks for paving purposes has been a subject of much discussion, and a great variety of forms and dimensions are to be found in all cities.

For stability, a certain proportion must exist between the depth, the length, and the breadth. The depth must be such that when the wheel of a loaded vehicle passes over one edge of the upper surface of a block, the block will not tend to tip up. The resultant direction of the pressure of the load and adjoining blocks always should tend to depress the whole block vertically; where this does not happen, the maintenance of a uniform surface is impossible. To fulfill this requirement, it is not necessary to make the block more than 6 inches deep.

Width. The maximum width of blocks is controlled by the size of horses' hoofs. To afford good foothold to horses drawing heavy loads, it is necessary that the width of each block, measured along the street, shall be the least possible consistent with stability. If the width be great, a horse drawing a heavy load, attempting to find a joint, slips back, and requires an exceptionally wide joint to pull him up. It is therefore desirable that the width of a block shall not exceed 3 inches; or that four blocks, taken at random and placed side by side, shall not measure more than 14 inches.

Length. The length, measured across the street, must be sufficient to break joints properly, for two or more joints in line lead to the formation of grooves. For this purpose the length of the block should be not less than 9 inches nor more than 12 inches.

Form. The blocks should be well squared, and must not taper in any direction; sides and ends should be free from irregular projections. Blocks that taper from the surface downwards (wedge-shaped) should not be permitted in the work; but if any are allowed, they should be set with the widest side down.

Manner of Laying Blocks. The blocks should be laid in parallel courses, with their longest side at right angles to the axis of the formed. This pavement offers less resistance to traction than cobblestones, but it is almost equally rough and noisy.

Granite-Block Pavement. The Belgian block gradually has been displaced by the introduction of rectangular blocks of granite. Blocks of comparatively large dimensions were employed at first. They were from 6 to 8 inches in width on the surface, from 10 to 20 inches in length, with a depth of 9 inches. They merely were placed in rows on the subsoil, perfunctorily rammed, the joints filled with sand, and the street thrown open to traffic. The unequal settlement of the blocks, the insufficiency of the foothold, and the difficulty of cleansing the street, led to the gradual development of the latest type of stone-block pavement, which consists of narrow, rectangular blocks of granite, properly proportioned, laid on an unyielding and impervious foundation, with the joints between the blocks filled with an impermeable cement.

Experience has proved beyond doubt that this latter type of pavement is the most enduring and economical for roadways subjected to heavy and constant traffic. Its advantages are many, while its defects are few.

Advantages.

(1) Adapted to all grades.
(2) Suits all classes of traffic.
(3) Exceedingly durable.
(4) Foothold, fair.
(5) Requires but little repair.
(6) Yields but little dust or mud.
(7) Facility for cleansing, fair.

Defects.

(1) Under certain conditions of the atmosphere, the surface of the pavement becomes greasy and slippery.

(2) The incessant din and clatter occasioned by the movement of traffic is an intolerable nuisance; it is claimed by many physicians that the noise injuriously affects the nerves and health of persons who are obliged to live or do business in the vicinity of streets so paved.

(3) Horses constantly employed upon it soon suffer from the continual jarring produced in their legs and hoofs, and quickly wear out.

(4) The discomfort of persons riding over the pavement is very great, because of the continual jolting to which they are subjected.

(5) If stones of an unsuitable quality are used—for example, those that polish—the surface quickly becomes slippery and exceedingly unsafe for travel.

Blocks. *Size and Shape.* The proper size of blocks for paving purposes has been a subject of much discussion, and a great variety of forms and dimensions are to be found in all cities.

For stability, a certain proportion must exist between the depth, the length, and the breadth. The depth must be such that when the wheel of a loaded vehicle passes over one edge of the upper surface of a block, the block will not tend to tip up. The resultant direction of the pressure of the load and adjoining blocks always should tend to depress the whole block vertically; where this does not happen, the maintenance of a uniform surface is impossible. To fulfill this requirement, it is not necessary to make the block more than 6 inches deep.

Width. The maximum width of blocks is controlled by the size of horses' hoofs. To afford good foothold to horses drawing heavy loads, it is necessary that the width of each block, measured along the street, shall be the least possible consistent with stability. If the width be great, a horse drawing a heavy load, attempting to find a joint, slips back, and requires an exceptionally wide joint to pull him up. It is therefore desirable that the width of a block shall not exceed 3 inches; or that four blocks, taken at random and placed side by side, shall not measure more than 14 inches.

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Form. The blocks should be well squared, and must not taper in any direction; sides and ends should be free from irregular projections. Blocks that taper from the surface downwards (wedge-shaped) should not be permitted in the work; but if any are allowed, they should be set with the widest side down.

Manner of Laying Blocks. The blocks should be laid in parallel courses, with their longest side at right angles to the axis of the

Figs. 79 and 80. The reason for this is to prevent the formation of longitudinal ruts, which would happen if the blocks were laid lengthwise. Laying blocks obliquely and "herringbone" fashion has been tried in several cities, with the idea that the wear and formation of ruts would be reduced by having the vehicle cross the blocks diagonally. The method has failed to give satisfactory results; the wear was irregular and the foothold defective; the difficulty of construction was increased by reason of labor required to form the triangular joints; and the method was wasteful of material.

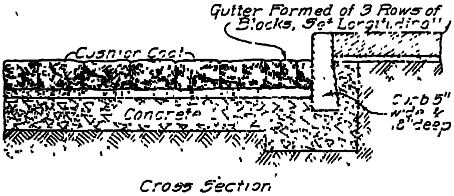

Fig. 79. Section Showing Method of Laying Stone-Blocks

The gutters should be formed by three or more courses of block, laid with their length parallel to the curb.

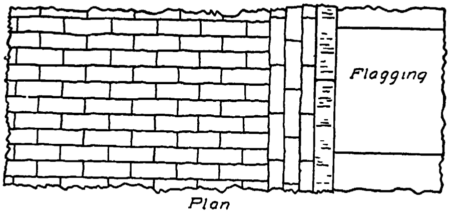

Fig. 80. Plan of Stone-Block Pavement Showing Method of Laying Blocks

At junctions or intersections of streets, the blocks should be laid diagonally from the center, as shown in Fig. 81. The reasons for this are: (1) to prevent the traffic crossing the intersection from following the longitudinal joints and thus forming depressions and ruts; (2) laid in this manner, the blocks afford a more secure foothold for horses turning the corners. The ends of the diagonal blocks where they abut against the straight blocks, must be cut to the required bevel.

The blocks forming each course must be of the same depth, and no deviation greater than $\frac{1}{4}$ inch should be permitted. The blocks should be assorted as they are delivered, and only those corresponding in depth and width should be used in the same course. The better method would be to gage the blocks at the quarry. This would lessen the cost considerably; it would avoid also the inconvenience to the public due to the stopping of travel because of the rejection of defective material on the ground. This method undoubt edly would be preferable to the contractor, who would be saved expense of handling unsatisfactory material; and it also would le the inspectors free to pay more attention to the manner in which work of paving is performed.

The accurate gaging of the blocks is a matter of much im tance. If good work is to be executed, the blocks, when laid, mus in parallel and even courses; and if the blocks are not gaged accurat to one uniform size, the result will be a badly paved street, with courses running unevenly. The cost of assorting blocks into lot uniform width, after delivery on the street, is far in excess of

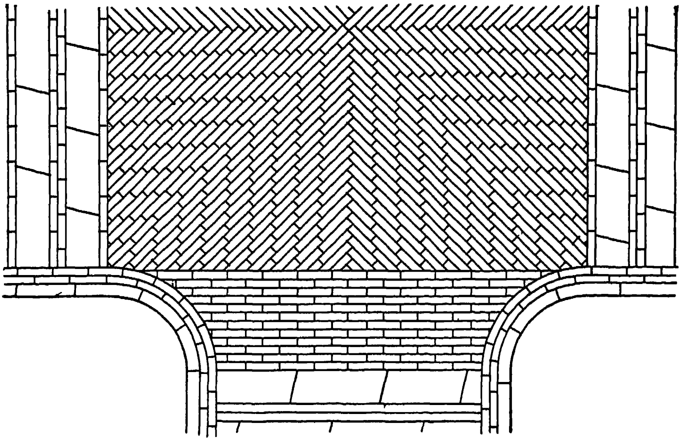

Fig. 81. Diagram Showing Method of Laying Stone Blocks at Intersection of Streets

additional price which would have to be paid for accurate gaging the quarry.

Foundation. The foundation of the blocks must be solid a unyielding. A bed of hydraulic-cement concrete is the most suital and its thickness must be regulated according to the traffic; t thickness, however, should not be less than 4 inches, and need not more than 9 inches. A thickness of 6 inches will sustain traffic 600 tons per foot of width.

Cushion Coat. Between the surface of the concrete and the b of the blocks, there must be placed a cushion coat formed of incompressible but mobile material, the particles of which read and transfer the pressure of the traffic uniformly to the concrete below. A layer of dry, clean sand 1 inch to 2 inches thick forms an excellent cushion coat. Its particles must be of such fineness as to pass through a No. 8 screen; if the sand is coarse and contains pebbles, it will not adapt itself to the irregularities of the bases of the blocks; hence the blocks will be supported at a few points only, and unequal settlement will take place when the pavement is subjected to the action of traffic. The sand also must be perfectly free from moisture, and artificial heat must be used to dry it if necessary. This requirement is an absolute necessity. There should be no moisture below the blocks when laid; nor should water be allowed to penetrate below the blocks; if such happens, the effect of frost will be to upheave the pavement and crack the concrete.

Where the best is desired without regard to cost, a layer of asphaltic cement ½ inch thick may be substituted for the sand, with superior and very satisfactory results.

Laying Blocks. The blocks should be laid stone to stone, so that the joint may be of the least possible width; wide joints cause increased wear and noise, and do not increase the foothold. The courses should be commenced on each side and should be worked toward the middle; and the last stone should fit tightly.

Ramming. After the blocks have been set, they should be well rammed down; and the stones which sink below the general level should be taken up and replaced with a deeper stone or brought to level by increasing the sand bedding.

The practice of workmen invariably is to use the rammer so as to secure a fair surface. This does not give the result intended to be secured, but brings each block to an unyielding bearing. The result of such a surfacing process is to produce an unsightly and uneven roadway when the pressure of traffic is brought upon it. The rammer used should weigh not less than 50 pounds and have a diameter of not less than 3 inches.

Fillings for Joints. All stone-block pavements depend for their waterproof qualities upon the character of the joint filling. Joints filled with sand and gravel of course are pervious. A grout of lime or cement mortar does not make a permanently waterproof joint, it becomes disintegrated under the vibration of traffic. An impervious joint can be made only by employing a filling made from bituminous or asphaltic material; this renders the pavement more impervious to moisture, makes it less noisy, and adds considerably to its strength.

Bituminous Cement for Joint Filling. The bituminous materials employed are: (1) coal tar having a specific gravity between 1.23 and 1.33 at 60 degrees Fahrenheit, a melting point between 120 and 130 degrees Fahrenheit, and containing not over 30 per cent of free carbon. (2) asphalt, either natural or artificial, entirely free from coal tar or any product of coal-tar distillation, and containing not less than 98 per cent of pure bitumen soluble in carbon bisulphide. Of the total amount soluble in carbon bisulphide, 98.5 per cent must be soluble in carbon tetrachloride. The penetration, when tested by the Dow method, must be not greater than 110, at 115 degrees Fahrenheit, and at 77 degrees Fahrenheit must range between 25 and 60. The specific gravity at 60 degrees Fahrenheit must not be more than 1.00.

The mode of applying the coal-tar filler is as follows: After the blocks are laid, gravel heated to about 250 degrees Fahrenheit is spread over the surface and swept into the joints until they are filled to a depth of about 2 inches. The blocks then are rammed. The coal-tar filler heated to a temperature between 250 and 300 degrees Fahrenheit is poured into the joints until they are about half filled, hot gravel is swept in until it reaches to within ½ inch of the surface, and hot filler is then poured in until it is flush with the surface of the blocks, after this sufficient hot gravel is applied to the joints to conceal the filler.

In applying the coal-tar filler it is essential that both the gravel and filler are heated sufficiently. Otherwise the filler will be chilled and will not flow to the bottom of the joint, but will form a thin layer near the surface, which under the action of frost and the vibration of traffic, will be cracked and broken up quickly; the gravel will settle, and the blocks will be jarred loose, causing the surface of the pavement to become a series of ridges and hollows. The filler should not be applied during a rainfall or while the blocks are wet or damp, for such a condition would prevent the filler from adhering to the blocks. The asphalt filler is heated to a temperature between 400 and 450 degrees Fahrenheit and poured into the joints until

Hydraulic-Cement Filler is composed of equal parts of Portland cement and sharp sand mixed with clean fresh water to a suitable consistency. The joints between the blocks are filled to a depth of 2 inches with gravel, and the blocks are rammed, after which the filler is poured into the joints until they are filled flush with the surface of the blocks. In dry weather the blocks should be moistened by sprinkling with water before applying the filler. After the filler has taken its initial set, the whole surface of the pavement is covered with a layer of sand about ½ inch thick and if the weather is dry and warm it is sprinkled with water daily for three days. Traffic is not permitted to use the pavement until at least seven days after completion.

Stone Pavement on Steep Grades. Stone blocks may be employed on all practicable grades, but on grades exceeding 10 per cent, cobblestones afford a better foothold than blocks. The cobblestones should be of uniform length, the length being at least twice the breadth, say stones 6 inches long and 2½ inches to 3 inches in diameter. These should be set on a concrete foundation, laid stone to stone, and the interstices filled with cement grout or bituminous cement, or a bituminous-concrete foundation may be employed and the interstices between the stones may be filled with asphaltic paving cement. Should stone blocks be preferred, they must be laid, when the grade exceeds 5 per cent, with a serrated surface, by either of the methods shown in Figs. 82 and 83. The method shown in Fig. 82 consists in slightly tilting the blocks on their bed so as to form a series of ledges or steps which will [illegible]

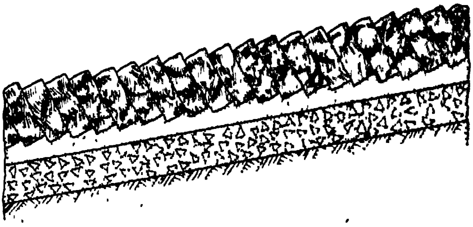

Fig. 82. Laying Stone Pavement on Steep Grades by Tilting Blocks.

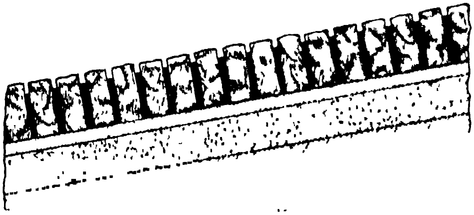

Fig. 83. Laying Stone Pavement on Steep Grade by Separating Blocks and Filling with Grout.

method shown in Fig. 83 consists in placing between the rows of stones a course of slate, or strips of creosoted wood, rather less than 1 inch in thickness and about 1 inch less in depth than the blocks; or the blocks may be spaced about 1 inch apart, and the joints filled with a grout composed of gravel and cement. The pebbles of the gravel should vary in size between $\frac{1}{4}$ inch and $\frac{3}{4}$ inch.

BRICK PAVEMENTS

A brick pavement consists of vitrified bricks laid on a suitable concrete foundation, Fig. 84.

Qualifications of Brick. The qualities essential to a good paving brick are the same as for any other paving material, viz, hardness, toughness, and ability to resist the disintegrating effects of water and frost. These qualities are imparted to the brick by a process of annealing, through which the clay is brought to the point of fusion, and the heat then gradually reduced until the kiln is cold.

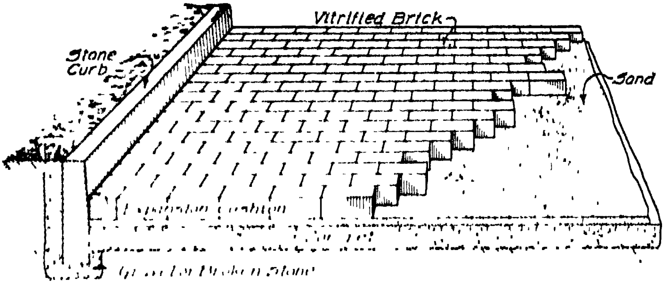

Fig. 84. Section Showing Method of Laying Vitrified Brick Pavement.

Composition. The material from which is made the majority of the brick used for paving is a shale. Shales are indurated clays with a laminated structure and the appearance of slate, and occur in stratified beds. The average composition of the shales that have proved satisfactory for the manufacture of paving brick is shown in Table XIV.

An excess of silica causes brittleness; or an excess of alumina causes shrinking, cracking, and warping. Iron renders the clay fusible and makes the brick more homogeneous. Lime in the form

TABLE XIV

Average Composition of Shales for Paving-Brick Manufacture

CONSTITUENTS	PROPORTIONAL PART (per cent)
(Non-Fluxing)	
Silica	56.0
Alumina	22.0
Water and loss on ignition	7.0
Moisture	2.0
(Fluxing)	
Sesquioxide of iron	7.0
Lime	1.0
Magnesia	1.0
Alkalies	4.0
Total	100.0

decrease the strength of the brick; at a high temperature it is changed into caustic lime, which, while rendering the clay more fusible, will absorb moisture upon exposure to the weather and thus cause the brick to disintegrate. Magnesia exerts but little influence on the character of the brick. The alkalies in small quantities render the clay fusible.

Color. The color of the clay is of no practical importance, it is due to the presence of the metallic oxides and organic substance Iron produces bricks which are either red, yellow, or blue, according to the quantity present and the degree of heat; some organic substances produce a blue, bluish-gray, or black color.

The color of the brick is governed partly by the color of the clay, by the temperature of burning, by the kind of fuel used, and by the sand that is used to prevent the brick from sticking to the dies or to each other in the kiln.

Manufacture. In the manufacture of the brick, the shale is crushed usually in dry pans and then passed through a 4-mesh or an 8-mesh screen. The screened material is mixed with water in a pug mill to the required consistency. The finer the material is crushed and the more thoroughly it is worked or tempered in the mill, the more uniform and better the brick is.

The plastic clay, in the "stiff-mud" process, as it leaves the pug mill is forced by an auger through a die which forms a bar of stiff clay of the desired dimensions, and this is cut by an automatic cutter into bricks of the size required. The bricks then, in some factories, are repressed in a die, during which the edges of the brick are rounded and the lugs, grooves, and trade-mark stamped on the sides. When repressing is not practiced, the bar of clay as it comes from the pug mill is cut by wires, the brick being called "wire-cut lug" brick.

The bricks, made by either method, are placed in a heated chamber to dry, this requiring from 18 to 60 hours according to the clay, temperature, and plant arrangement. When dry the bricks are stacked in the kiln, which is usually of the down-draft type with furnaces built in the outer walls. The bottom of the kiln is perforated to allow the gases to pass through to the flues placed below the floor and connected to the chimney. The heat from the furnace passes upward into the kiln, then downward through the bricks into the flues and thence to the chimney. At the beginning of the burning the heat is applied slowly to drive off the contained water without cracking the bricks. When the dryness of the smoke indicates the absence of moisture in the bricks, the fires are gradually increased until the temperature throughout the kiln is from 1500 to 2000 degrees Fahrenheit, this temperature being maintained from seven to ten days. The kiln then is closed, the fires are drawn, and the bricks are allowed to cool. This part of the process is called annealing, and to produce a tough brick requires from seven to ten days. The cooled bricks are sorted into different lots; the No. 1 paving bricks are generally found in the upper layers in the kiln.

Sizes. Two sizes of bricks are made: one size measuring 8½ × 2½ × 4 inches weighing about 7 pounds and requiring 58 to the square yard. The other, measuring 8½ × 3½ × 4 inches and frequently called "blocks", weighs about 9½ pounds and requires 45 to the square yard.

Characteristics. The characteristics of brick suitable for paving are: not to be acted upon by acids; shale bricks not to absorb more than 2 per cent nor less than ½ of 1 per cent of their weight of water, and clay bricks not to absorb more than 6 per cent of their weight of water (the absorption by a shale brick of less than ½ of 1 per cent of its weight of water, indicates that it has been overburned); when broken with a hammer, to show a dense close-grained structure, free spall, or chip, when quickly struck on the edges; hard but not brittle.

Tests for Paving Brick. To ascertain if brick possesses the required qualities they are subjected to three tests: (1) abrasion by impact (commonly called the "rattler" test); (2) absorption; (3) cross breaking.

The Rattler Test. The rattler is a steel barrel 28 inches long and 28 inches in diameter, the sides formed of 14 staves fastened to two cast-iron heads furnished with trunnions which rest in a cast-iron frame. It is provided with gears and a belt pulley arranged to revolve at a rate of from 29½ to 30½ revolutions per minute. The material employed to abrade the brick is spherical balls of cast iron, the composition of which is: combined carbon, not less than 2.50 per cent; graphitic carbon, not more than 0.10 per cent; silicon, not more than 1 per cent; manganese, not more than 0.50 per cent; phosphorus, not more than 0.25 per cent; sulphur, not more than 0.08 per cent. Two sizes of balls or shot are used, the larger being 3.75 inches in diameter when new and weighing about 7½ pounds, the smaller being 1.875 inches in diameter and weighing 0.95 pounds. A charge consists of ten large shot with enough small shot to make a weight of 300 pounds. The shot is used until the large size is worn to a weight of 7 pounds and the small shot is worn to a size that will pass through a circular hole 1¾ inches in diameter made in a cast-iron plate ¼-inch thick.

The brick to be tested are subjected to a temperature of 100 degrees Fahrenheit for three hours. Ten bricks are weighed and placed in the rattler with a charge of spherical shot, and the rattler is revolved for 1800 revolutions. The bricks then are taken out, pieces less than 1 pound in weight are removed and the balance weighed. From the weights before and after rattling the percentage of loss is calculated. The loss ranges from 16 per cent to 40 per cent. Brick to be used under heavy traffic should not lose more than 22 per cent, and for light traffic not more than 28 per cent.

Absorption Test. The absorption test is made on five bricks that have been through the rattler test. They are weighed, and are immersed in water for 48 hours, then are taken out and weighed, with the surplus water wiped off. From the weights before and after

Cross-Breaking Test. This test is made by placing a brick edge on supports 6 inches apart. The load is applied at the center of the brick, and is increased uniformly until fracture occurs. The average of the result on ten bricks is used in computing the modulus of rupture, $R=\frac{3WL}{2bd^2}$; in which W is the average breaking load in pounds, L the length between supports in inches, b the breadth, and d the depth in inches.

Brick-Pavement Qualifications. *Advantages.* The advantages of brick pavement may be stated as follows:

(1) Easy traction.
(2) Good foothold for horses.
(3) Not disagreeably noisy.
(4) Yields but little dust and mud.
(5) Adapted to all grades.
(6) Easily repaired.
(7) Easily cleaned.
(8) But slightly absorbent.
(9) Pleasing to the eye.
(10) Expeditiously laid.
(11) Durable under moderate traffic.

Defects. The principal defects of brick pavements arise from lack of uniformity in the quality of the bricks, and from the liability of incorporating in the pavement bricks too soft or too porous a structure, which crumbles under the action of traffic or frost.

Foundation. A brick pavement should have a firm foundation. As the surface is made up of small, independent blocks, each one must be supported adequately, or the load coming upon it may force it downwards and cause unevenness, a condition which conduces to the rapid destruction of the pavement. Several forms of foundation have been used such as gravel, plank, sand, broken stone, and concrete. The last mentioned is the best.

Sand Cushion. The sand cushion is a layer of sand placed on top of the concrete to form a bed for the brick. Practice regarding the depth of this layer of sand varies considerably. In some cases it is only ½ inch deep, varying from this up to 3 inches. The pavement less hard and rigid than would be the case were the brick laid directly on the concrete.

The sand is spread evenly, sprinkled with water, smoothed, and brought to the proper contour by screeds or wooden templets, properly trussed and mounted on wheels or shoes which bear upon the upper surface of the curb. Moving the templet forward levels and forms the sand to a uniform surface and proper shape.

The sand used for the cushion coat should be clean and free from loam, moderately coarse, and free from pebbles exceeding ¼ inch in size.

Manner of Laying. The bricks should be laid on edge or on one flat, as closely and compactly as possible, in straight courses across the street, with the length of the bricks at right angles to the axis of the street. Joints should be broken by at least 3 inches. None but whole bricks should be used, except in starting a course or making a closure. To provide for the expansion of the pavement, both longitudinal and transverse expansion joints are used, the former being made by placing a board templet ⅞-inch thick against the curb and abutting the brick thereto. The transverse joints are formed at intervals varying between 25 and 50 feet, by placing a templet or building lath ⅜-inch thick between two or three rows of brick. After the bricks are rammed and ready for grouting, these templets are removed, and the spaces so left are filled with coal-tar pitch or asphaltic paving cement. The amount of pitch or cement required will vary between 1 and 1½ pounds per square yard of pavement, depending upon the width of the joints. After 25 or 30 feet of the pavement is laid, every part of it should be rammed with a rammer weighing not less than 50 pounds and the brick which sink below the general level should be removed, sufficient sand being added to raise the brick to the required level. After all objectionable brick have been removed, the surface should be swept clean, then rolled with a steam roller weighing from 3 to 6 tons. The object of rolling is to bring the bricks to an unyielding bearing with a plane surface; if this is not done, the pavement will be rough and noisy and will lack durability. The rolling should be executed first longitudinally, beginning at the crown and working toward the gutter, taking care that each return trip of the roller covers exactly the same area as the preceding trip passage may neutralize any careening of the brick due to the first passage.

The manner of laying brick at street intersections is shown in Fig. 85.

Joint Fillings. The character of the material used in filling the joints between the brick has considerable influence on the success

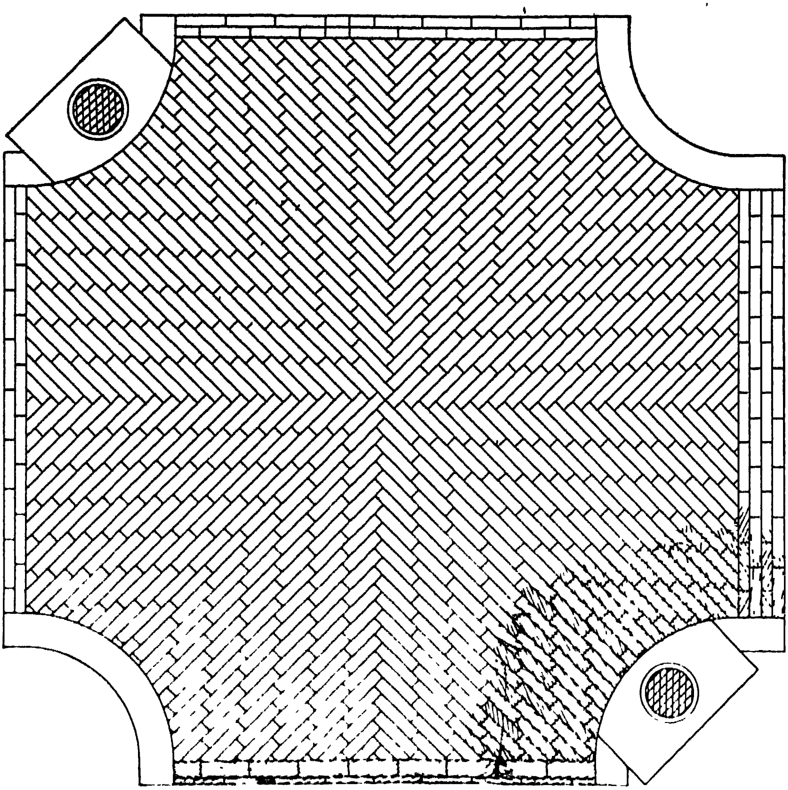

Fig. 85. Method of Laying Brick at Street Intersections

and durability of the pavement. Various materials have been used such as sand, coal-tar pitch, asphalt, mixtures of coal tar and asphalt, and Portland cement, besides various patented fillers, as "Murphy's grout", which is made from ground slag and cement. Each material has its advocates, and there is much difference of opinion as to which gives the best results.

The best results seem to be obtained by using a high grade of composition; the presence of the lime increasing the tendency of the filler to swell through absorption of moisture, causing the pavement to rise or to be lifted away from its foundation, and thus producing the roaring or rumbling noise so frequently complained of.

The Portland-cement grout, when uniformly mixed and carefully placed, resists the impact of traffic and wears well with brick. When a failure occurs, repairs can be made quickly, and, if made early, the pavement will be restored to a good condition. If, however, repairs are neglected, the brick soon loosen and the pavement fails.

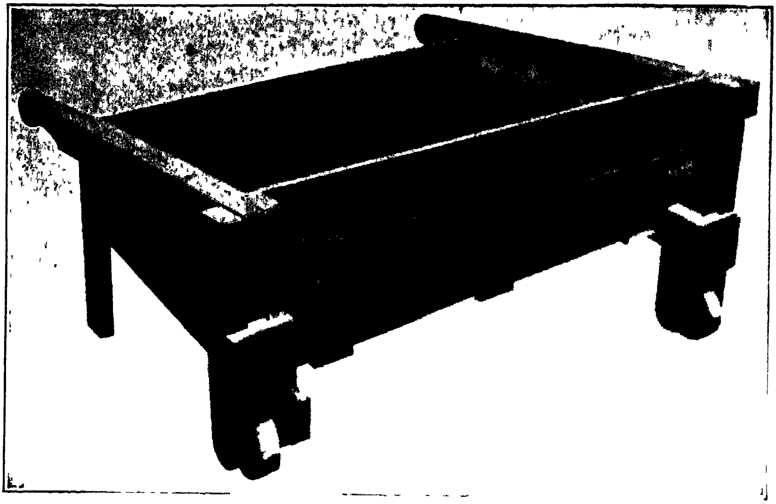

Fig. 86. Grout Box Used in Laying Brick Pavement
Courtesy of National Paving Brick Manufacturers [illegible]

The office of a filler is to prevent water from reaching the foundation, and to protect the edges of the brick from spalling under traffic. In order to meet both of these requirements, every joint must be filled to the top, and must remain so, wearing down with the brick. Sand does not meet these requirements. Although at first making a good filler, being inexpensive and reducing the liability of the pavement to be noisy, it soon washes out, leaving the edges of the brick unprotected and consequently liable to be chipped. Coal tar and the mixtures of coal tar and asphalt have an advantage in rendering a pavement less noisy and in cementing together any breaks that may but, unless made very hard, they have the disadvantage of becoming soft in hot weather and flowing to the gutters and low places in the pavement, there forming a black and unsightly scale and leaving the high parts unprotected. The joints, thus deprived of their filling, become receptacles for water, mud, and ice in turn; and the edges of the brick are broken down quickly. Some of these mixtures become so brittle in winter that they crack and fly out of the joints under the action of traffic.

The Portland-cement filler is prepared by mixing 2 parts of cement and 1 part of fine sand with sufficient water to make a thin grout. The most convenient arrangement for preparing and distributing the grout is a water-tight wooden box carried on four wood wheels about 12 inches in diameter, Fig. 86. The box may be about 4 feet wide, 7 feet long, and 12 inches deep, furnished with a gate about 8 inches wide, in the rear end. The box should be mounted on the wheels with an inclination, so that the rear end is about 4 inches lower than the front end.

Following are the successive operations of placing the filler: The cement and sand are placed in the box, and sufficient water is added to make a thin grout. The grouting box is located about 12 feet from the gutter, the end gate opened, and about 2 cubic feet of the grout allowed to flow out and run over the top of the brick (care being taken to stir the grout while it is being discharged), Fig. 87. If the brick are very dry, the entire surface of the pavement should be wet thoroughly with a hose before applying the grout; if not, absorption of the water from the grout by the bricks will prevent adhesion between the bricks and the cement grout. The grout is swept into the joints by ordinary bass brooms. After a length of about 100 feet of the pavement has been covered the box is returned to the starting point, and the operation is repeated with a grout somewhat thicker than the first. If this second application is not sufficient to fill the joints, the operation is repeated as often as may be necessary to fill them. If the grout has been made too thin, or the grade of the street is so great that the grout will not remain long enough in place to set, dry cement may be sprinkled over the joints and swept in. After the joints are filled completely and inspected, allowing three or four hours to intervene.

[illegible] Applying Filler to Brick Pavement
[illegible] Manufacturers Association, Cleveland, Ohio

of about ½ inch, and the roadway barricaded, and no traffic allowed on it for at least ten days.

The object of covering the pavement with sand is to prevent the grout from drying or settling too rapidly; hence, in dry and windy weather, it should be sprinkled from time to time. If coarse sand is employed in the grout, it will separate from the cement during the operation of filling the joints, with the result that many joints will be filled with sand and very little cement, while others will be filled with cement and little or no sand; thus there will be many spots in the pavement in which no bond is formed between the bricks, and under the action of traffic these portions quickly will become defective.

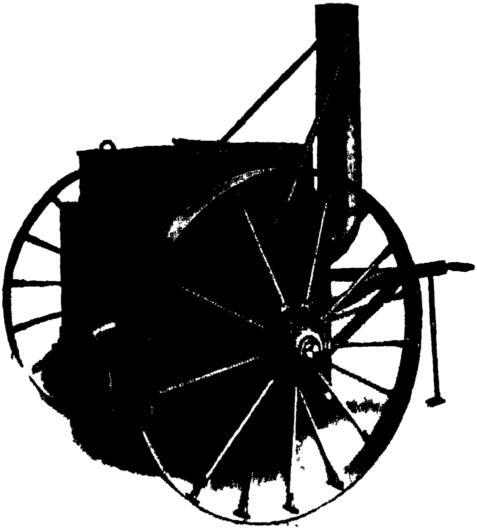

Fig. 88. Coal Tar Heating Tank
Courtesy of Barber Asphalt Paving Company, Philadelphia, Pennsylvania

The coal-tar filler is best applied by pouring the material from buckets, and brooming it into the joints with wire brooms, and in order to fill the joints effectually, it must be used only when very hot. To secure this condition, a heating tank on wheels is necessary, Fig. 88. It should have a capacity of at least 5 barrels, and be kept at a uniform temperature all day. One man is necessary to feed

[illegible] Located in City. Eight Years Old When Picture Was Taken

[illegible] Brick Manufacturers Association, Cleveland, Ohio

the buckets from the heating tank to a third, who pours the material over the street. The latter starts to pour in the center of the street, working backward toward the curb, and pouring a strip about 2 feet in width. A fourth man, with a wire broom, follows immediately after him, sweeping the surplus material toward the pourer and in the direction of the curb. This method leaves the entire surface of the pavement covered with a thin coating of pitch, which immediately should be covered with a light coating of sand, the sand becoming imbedded in the pitch. Under the action of traffic, this thin coating is worn away quickly, leaving the surface of the bricks clean and smooth, Fig. 89.

Tools Used by Hand in the Construction of Block Pavements. The principal tools required in constructing block pavements comprise *hammers* and *rammers* of varying sizes and shapes, depending on the material and size of the blocks to be laid; also *crowbars*, sand *screens*, and rattan and wire *brooms*. Cobblestones, square blocks, and brick require different types both of hammer and rammers for adjusting them to place and for forcing them to their seats. A cobblestone rammer, for example, is usually made of wood (generally locust) in the shape of a long truncated cone, banded with iron at top and bottom, weighing about 40 pounds, and having two handles, one at the top and another on one side. A Belgian-block rammer is slightly heavier, consisting of an upper part of wood set in a steel base; while a rammer for granite blocks is still heavier, comprising an iron base with cast-steel face, into which is set a locust plug with hickory handles. For laying brick, a wooden rammer shod with cast iron or steel and weighing about 27 pounds is used. A light rammer of about 20 pounds weight, consisting of a metallic base attached to a long, slim, wooden handle, is used for miscellaneous work, such as tamping in trenches, next to curbs, etc.

Concrete Mixing Machine. Where large quantities of concrete are required, as in the foundations of improved pavements, concrete can be prepared more expeditiously and economically by the use of mechanical mixers, and the ingredients will be mixed more thoroughly than by hand. Thorough incorporation of the ingredients is an essential element in the quality of a concrete. When mixed by depends upon the proper manipulation of the hoe and shovel. The manipulation, although extremely simple, is rarely performed by the ordinary laborer as it should be unless he is watched constantly by the overseer.

Several varieties of concrete-mixing machines are in the market, all of which are efficient and of good design. A convenient portable type is illustrated in Fig. 90. The capacity of the mixer ranges from 5 to 20 cubic yards per hour, depending upon the regularity with which the materials are supplied, speed, etc.

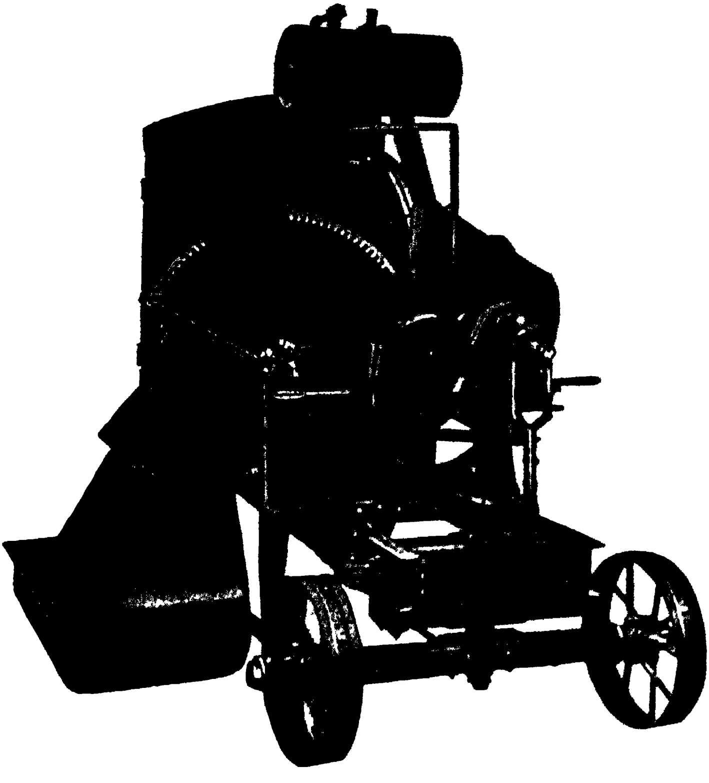

Fig. 90. Smith Concrete Mixer on Truck with Gasoline Engine, Power Charger and Water Tank.
Courtesy of T. L. Smith Company, M[illegible]

Gravel Heaters. A special type of oven usually is employed for heating the gravel used for joint filling in stone-block pavements. These heaters are made in various sizes, a common size being 9 feet [illegible]

WOOD-BLOCK PAVEMENTS

Wood-block pavements, Fig. 91, are formed of rectangular blocks measuring from 3½ to 4 inches wide, 5 to 10 inches long, and 4 inches deep, impregnated with creosote, or other preservative, laid in a bed of Portland-cement mortar spread upon a concrete foundation, with the joints between the blocks filled with either Portland-cement grout, or a bituminous filler.

The wood used is obtained from the long-leaved yellow pine (pinus palustrus), lob-lolly pine (pinus taeda), short-leaved pine (pinus echinata), Cuba pine (pinus heterophylla), black gum (nyssa sylvatica), red gum (liquidambar styrraciflua), Norway pine (pinus resinosa), or tamarack (larix laricina).

The wood should be cut from sound trees, free from cracks, snakes, and knots.

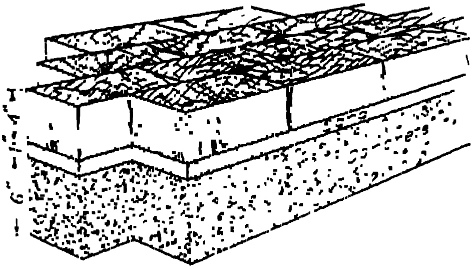

Fig. 91 Section Showing Foundation for Wood-Block Pavements and Method of Laying Blocks

The great enemy of wood pavement is decay due to a low form of plant life called fungi. The fungi attack the wood from the outside, and if the wood is in the right condition for the spores to grow, they ultimately will penetrate the entire structure of the wood. There are three classes of fungi: one which attacks all parts of the wood structure; another which attacks the cellulose; and a third, which is the most common, and attacks only the lignin the name of the many organic substances that are incrusted around the cellulose, and which with the latter constitute the essential part of woody tissue here the fungi dissolve the lignin and the cellulose to make food for their development. Heat, air, and moisture are necessary to the existence of the fungi; without any one of these elements they cannot live. To destroy the fungus life and preserve wood from decay many processes have been devised, the one that seems to meet the requirements better than any other is the process of creosoting.

Creosoting. This process consists in impregnating the wood has been removed. Its effect on the wood is to coagulate the albumen and thereby prevent its decomposition, also to fill the pores of the wood with a bituminous substance which excludes both air and moisture, and which is obnoxious to the lower forms of animal and vegetable life.

The coal-tar creosote oil is used without admixture or adulteration with other oils or tars. Its characteristics are: specific gravity, 1.03 to 1.08, at a temperature of 100 degrees Fahrenheit; contain not more than 5 per cent of tarry matter, nor more than 2 per cent of water, and not more than 8 per cent of tar acids, 99 per cent to be soluble in hot benzol; when subjected to distillation at gradually increasing temperatures up to 400 degrees Fahrenheit not more than 5 per cent of distillate shall pass over, at 450 degrees not more than 35 per cent, and up to 600 degrees Fahrenheit not more than 80 per cent; after complete distillation not more than 2 per cent of coke shall remain; upon sulphonating a sample of the total distillate, the residue shall not exceed 1 per cent.

For applying the creosote to the wood, several methods are followed. The one in most favor for paving blocks is the "pressure process", which essentially consists in: (1) steaming the wood for the purpose of liquefying the sap and other substances contained in the interfibrous spaces; (2) creating a vacuum for the purpose of removing the liquefied substances; (3) injecting the creosote under pressure.

The operation is performed in metal cylinders called "retorts", 6 or more feet in diameter and of any desired length, usually about 100 feet. The load of blocks, called a "charge", is placed upon metal cars called "buggies" and is run into the retort cylinder, the ends of which then are hermetically closed with "heads" or doors. Steam, at a gage pressure varying from 15 pounds to 45 pounds per square inch, is admitted to the retort (in some plants a vacuum is first created) and the pressure maintained for several hours. When the operator considers that the steaming has been continued a sufficient length of time, the products of condensation are removed from the retort through a blow-off cock in the bottom, when this is accomplished an air exhaust, or vacuum pump is put in operation, and a vacuum of from 20 inches to 26 inches is created and maintained for about one hour, at the end of which time to flow into the retort until it is filled. A pressure pump then is started to force the creosote into the retort until the pressure reaches 100 pounds to 150 pounds per square inch. This pressure is maintained until the required amount of creosote has been injected in the wood, then the surplus is drawn off, the heads opened, and the charge withdrawn.

The amount of creosote injected into the wood varies from 10 pounds to 22 pounds per cubic foot of wood. The amount is determined primarily by measuring the tanks and is verified by testing sample blocks. A sample block is bored entirely through in the direction of the fiber with an auger 1 inch in diameter, the hole being located midway between the sides and about $\frac{1}{3}$ the length of the block from one end. The borings are collected, thoroughly mixed, and the quantity and ratio of creosote to wood in the borings determined by extracting the creosote completely with carbon bisulphide.

The condition of the wood at the time of the treatment, is preferably dry and free from an excess of water. After treatment, and until used, the blocks during dry weather should be sprinkled frequently with water to prevent drying and cracking. The treated blocks are sometimes subjected to tests to determine the resistance to wear when saturated with water, the resistance to compression and impact, and to ascertain the amount of water the wood will absorb.

Laying the Blocks. The surface of the concrete foundation is cleaned from dust and dirt by sweeping, then sprinkled with water. Upon the cleaned surface a cushion coat is formed, by spreading a layer of sand 1 inch thick, Fig. 92, or a mortar composed of 1 part Portland cement and 2 parts sand, mixed with sufficient water to form a stiff paste (the practice of using a mixture of cement and sand lightly moistened with water produces a defective pavement). The blocks are set upon the cushion coat with the fiber vertical, Fig. 93, in straight, parallel courses at right angles to the axis of the street, except at intersections where they are set at an angle of 45 degrees with the axis of the street. They are laid so as to have the least possible width of joint (wide joints hasten the destruction of the wood by permitting the fibres to broom and wear under traffic).

Fig. 92. Spreading Sand Foundation for Wood Blocks in LaSalle Street, Chi
Courtesy of Engineering News, New York City

Fig. 93. Laying Wood Blocks in LaSalle Street, Chicago
Courtesy of Engineering News, New York City

Fig. 91. Wood-Block Pavement Being Hammered and Rolled, Preparatory to Putting in Filler
Courtesy of Engineering News, New York City

curb it is customary to place one or two rows of blocks with the length parallel to the curb and ¾ inch therefrom.

After the blocks are laid they are brought to a uniform surface by ramming with hand rammers or rolling with a light steam roller, Fig. 94. When laid upon a mortar cushion, the rolling or ramming must be completed before the mortar sets.

In some cases the cushion coat is omitted, the surface of the concrete freed from dust by dry sweeping is covered with a thin coat of a bituminous cement and the blocks laid directly upon it. Sometimes, the side and one end of each block, when it is about to be set in place, are dipped in the same bituminous material that is used to cover the concrete, the blocks are placed in contact and the surface is covered with a thin coating of the bituminous material, this being covered with a layer of sand or fine gravel.

After the blocks have been brought to a uniform surface, the joints are filled with either fine sand, cement grout, or a bituminous cement, Fig. 95. When sand is used, it should be fine and dry, spread over the surface of the pavement, and swept about until the joints are filled. Cement grout is made of equal parts of Portland cement and fine sand mixed with water to the required consistency. It is spread over the surface of the blocks and swept into the joints until they are filled. The surface of the pavement then is covered with sand, and the grout is allowed to set for about seven days before traffic is admitted. The bituminous filler is composed of coal-tar pitch, asphalt, or combinations of these, and other ingredients. The filler is applied hot in the same manner as described under brick pavement. To provide for the expansion of the blocks the joint next the curb is filled with bituminous filler.

Qualifications of Wood Pavements. *Advantages.* The advantages of wood pavement may be stated as follows:

(1) It affords good foothold for horses.

(2) It offers less resistance to traction than stone, and slightly more than asphalt.

(3) It suits all classes of traffic.

(4) It may be used on grades up to 5 per cent.

(5) It is moderately durable.

(6) It yields no mud when laid upon an impervious foundation.

(7) It yields but little dust.

(8) It is moderate in first cost.

(9) It is not disagreeably noisy.

Defects. The principal objections to wood pavement are:

(1) It is difficult to cleanse.

(2) Under certain conditions of the atmosphere it becomes greasy and very unsafe for horses. This may be remedied by covering the surface with a thin layer of fine sand or gravel; a similar treatment will absorb the oil which exudes during warm weather.

(3) It is not easy to open for the purpose of gaining access to underground pipes, it being necessary to remove rather a large surface for this purpose, which has to be left a little time after being repaired before traffic again is allowed upon it.

ASPHALT PAVEMENTS

Sheet=Asphalt Pavement. Sheet asphalt is the name used to describe a pavement having a wearing surface composed of sand graded in predetermined proportions, of a fine material or filler, and of asphalt cement, all incorporated by mixing in a mechanical mixer, and laid upon a concrete foundation, the surface of the latter being covered with a thin layer of bituminous concrete called a "binder".

Asphalt Cement. This is prepared from solid bitumen, refined and fluxed with (1) the residuum from paraffine petroleum; (2) the residuum from asphaltic petroleum; (3) a mixture of paraffine and asphaltic petroleum residuums; (4) natural malthas, or is prepared from (5) solid residual bitumen produced in the distillation of asphaltic petroleums, and fluxed with residuum oil produced from the same material.

Refined asphalt is that freed from the combined water and accompanying inorganic and organic matter. By comparatively simple operations the several varieties of asphalt may be separated from their impurities. Two methods are employed for refining; one using steam and the other direct fire. In both methods the asphalt is placed in tanks and slowly heated until thoroughly melted, and during the melting the mass is agitated by a current of either air or dry steam. The method of using steam is superior to the fire method. In the latter method there always is danger of overheating, in addi-

The varieties of asphalt known as gilsonite and grahamite, which are practically pure bitumen, do not require refining, but they are used to a very small extent in paving.

The greater part of the solid bitumen used for paving in the United States is obtained from the West Indies and South America. The more extensively used being that found at Trinidad, W. I., and at Bermudez, Venezuela. The asphalts known by the trade names "california" and "texaco" are produced by refining asphaltic oils, and may or may not require to be fluxed.

Fluxes are fluid oils and tars which are mixed with asphalt to produce a desired consistency. The refined asphalt is melted and the flux previously heated added to it, in the proportion required to produce the desired consistency. The mixture of asphalt and flux is agitated either by mechanical means or by a blast of air until the materials are thoroughly incorporated and the desired consistency is obtained.

Sand. The sand should be siliceous and so free from organic matter, mica, soft grains, and other impurities, that these will not amount to more than 2 per cent of its volume.

Fine Material or Filler. This consists of any sound stone, usually limestone or sand, pulverized to such fineness that the whole will pass the No. 50 sieve, and not more than 10 per cent will be retained on the No. 100 sieve, and at least 70 per cent will pass the No. 200 sieve. Portland cement sometimes is used instead of the pulverized stone.

The paving composition is prepared by heating the ingredients separately to a temperature between 300 and 350 degrees Fahrenheit, then incorporating them by mixing in a mechanical mixer. The hot sand is measured into the mixer, followed by the hot filler; these two materials are thoroughly mixed together, and the hot cement is added in such a way as to distribute it evenly over the mixed sand and filler; the mixing then is continued until the materials form a uniform and homogeneous mass, with the grains of sand completely covered with cement. A typical mixture is: sand 100 pounds; filler 17.5 pounds; bitumen in asphalt cement 17.5 pounds.

The proportions of the ingredients in the paving mixture are not constant, but vary with the climate of the place where the pavement is to be used, the character of the sand, and the amount and character of the traffic that will use the pavement. The ranges are indicated in the following data:

Data for Asphaltic Paving

Asphaltic Paving Mixture.

Constituents	Per Cent
Asphalt cement	12 to 15
Sand	70 to 83
Stone dust	5 to 15

Weight of Material. A cubic yard of the prepared material weighs about 4500 pounds. One ton of refined asphaltum makes about 2300 pounds of asphalt cement, equal to about 3.4 cubic yards of surface material.

Wearing Surface per Cubic Yard of Material.

Thickness (inches)	Area (sq yd)
2½	12
2	18
1½	27

Laying the Pavement. The hot paving mixture is hauled to the street and dumped at a place outside of the space in which it is to be laid. It then is thrown into place with hot shovels, and spread with hot rakes uniformly to such a depth as will give the required thickness when compacted. The finished thickness varies between 1½ inches and 2 inches. The reduction of thickness by compression is about 40 per cent generally. Before the mixture is spread, the surfaces of curbs and street fittings that will be in contact with it are painted with hot asphalt cement.

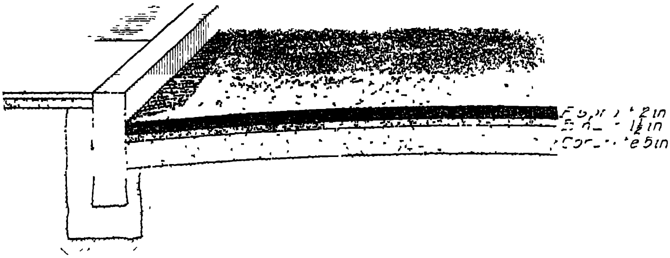

Fig. 96. Section of Asphalt Pavement Showing Layers

The pavement is constructed in two forms: (1) The paving

(2) the surface of the concrete foundation is covered with a coat of asphaltic concrete, Fig. 96, called the "binder course", the object of which is to unite more securely the wearing surface to the foundation. This it does by containing a larger percentage of cement, which, if put in the surface mixture, would render it too soft. The binder is composed of sound, hard stone broken to pass a 1¼-inch screen, sand, pulverized stone, and asphalt cement, mixed in the desired proportions. A typical mixture is: stone 100 pounds; sand 40 pounds; stone dust 8 pounds; bitumen in asphalt cement 8 pounds.

The paving composition is compressed by means of rollers and tamping irons, the latter being heated in a fire contained in an iron basket mounted on wheels. These irons are used for tamping such portions as are inaccessible to the roller, namely, gutters, around manhole heads, etc.

Two rollers are sometimes employed; one, weighing 5 to 6 tons and of narrow tread, is used to give the first compression; and the other, weighing about 10 tons and of broad tread, is used for finishing. The rate of rolling varies; the average is about 1 hour for 1000 square yards of surface. After the primary compression, natural hydraulic cement, or any impalpable mineral matter, is sprinkled over the surface, to prevent the adhesion of the material to the roller and to give the surface a more pleasing appearance. When the asphalt is laid up to the curb, the surface of the portion forming the gutter is painted with a coat of hot cement.

Although asphaltum is a poor conductor of heat, and the cement retains its plasticity for several hours, occasions may and do arise through which the composition before it is spread has cooled; its condition when this happens is analogous to hydraulic cement which has taken a "set", and the same rules which apply to hydraulic cement in this condition should be respected in regard to asphaltic cement.

If the temperature of the air at the time of hauling is below 70 degrees Fahrenheit the wagons carrying it are covered with canvas or other material to prevent the loss of heat. The temperature when delivered at the place where it is to be used must not be less than 280 degrees Fahrenheit.

Two methods are followed in laying an asphalt pavement adjoining street railway tracks: (1) a course of granite-block or brick paving is laid between the rail and the edge of the asphalt; (2) the asphalt is laid directly against the rail, which, if its temperature is below 50 degrees Fahrenheit, is heated by suitable apparatus to a temperature of about 60 degrees Fahrenheit immediately before the asphalt is laid.

Foundation. A solid, unyielding foundation is indispensable with all asphaltic pavements, because asphalt of itself has no power of offering resistance to the action of traffic, consequently it nearly always is placed upon a bed of hydraulic-cement concrete. The concrete must be set thoroughly and its surface dry before the asphalt is laid upon it; if not, the water will be sucked up and converted into steam, with the result that coherence of the asphaltic mixture is prevented, and, although its surface may be smooth, the mass is really honeycombed, so that as soon as the pavement is subjected to the action of traffic, the voids or fissures formed by the steam appear on the surface, and the whole pavement is broken up quickly.

Qualifications of Asphalt Pavements. *Advantages.* These may be summed up as follows:

(1) It gives easy traction.

(2) It is comparatively noiseless under traffic.

(3) It is impervious.

(4) It is easily cleansed.

(5) It produces neither mud nor dust.

(6) It is pleasing to the eye.

(7) It suits all classes of traffic.

(8) There is neither vibration nor concussion in traveling over it.

(9) It is laid expeditiously, thereby causing little inconvenience to traffic.

(10) Openings to gain access to underground pipes are easily made.

(11) It is durable.

(12) It is repaired easily.

Defects. These are as follows:

(1) It is slippery under certain conditions of the atmosphere

The American asphalts are much less so than the European, on account of their granular texture derived from the sand. The difference is very noticeable; the European are as smooth as glass, while the American resemble fine sandpaper. The slipperiness can be decreased by heating the surface of the pavement with a surface heater, then applying a layer of coarse sand and rolling it into the surface.

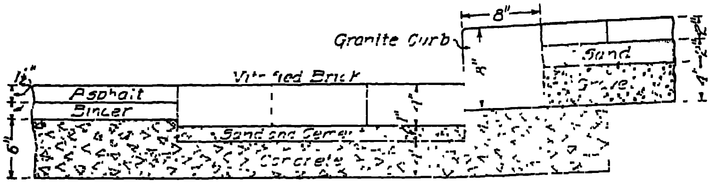

Fig. 97. Section of Asphalt Pavement Showing Use of Vitrified Brick to Form Gutter

(2) It will not stand constant moisture, and will disintegrate if sprinkled excessively.

(3) Under extreme heat it is liable to become so soft that it will roll or creep under traffic and present a wavy surface; and under extreme cold there is danger that the surface will crack and become friable.

(4) It is not adapted to grades steeper than 2½ per cent, although it is in use on grades up to 7.30 per cent.

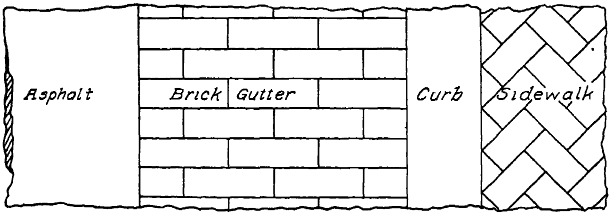

Fig. 98 Plan of Asphalt Pavement Showing Use of Vitrified Brick to Form Gutter

(5) Repairs must be made quickly, for the material has little coherence, and if, from irregular settlement of foundation or local violence, a break occurs, the passing wheels rapidly shear off the sides of the hole, and it soon assumes formidable dimensions.

Although pure asphaltum is impervious absolutely and insoluble in either fresh or salt water, yet asphalt pavements in the continued presence of water are quickly disintegrated. Ordinary rain or daily sprinkling does not injure them when they are allowed to become perfectly dry again. The damage is most apparent in gutters and adjacent to overflowing drinking fountains. This defect has long been recognized, and various measures have been taken to overcome it, or at least reduce it to a minimum. In some cities, ordinances have been passed, seeking to regulate the sprinkling of the streets; and in many places the gutters are laid with stone or vitrified brick, Figs. 97 and 98, while in others the asphalt is laid to the curb, a space of 12 to 15 inches along the curb being covered with a thin coating of asphalt cement.

Failure of Asphalt Pavement. The failure of asphalt pavement is due to any one, or a combination, of the following causes:

(1) *Unsuitable Materials.* The asphalt may be changed so by natural causes as to possess little or no cementing power. The fluxing agent may form only a mechanical instead of a chemical union with the asphalt, or its character may be such as to render the asphalt brittle, in which condition it easily is broken up under traffic. The sand may be graded improperly, either too coarse or too fine, or contain loam, vegetable matter, or clay.

(2) *Improper Manipulation.* The crude asphalt may have been refined at too high a temperature, which reduces or destroys the cementing property. The cement may be of improper consistence, of insufficient quantity, or inadequately mixed. If the cement is too hard, the pavement will have a tendency to crack during cold weather; and if too soft, it will push out of place and form waves under traffic. The quantity of cement needed varies with the character of the sand—a fine sand requires more cement than a coarse one, and the proportion of cement must be varied to suit the sand. When the ingredients are mixed inadequately, the cement and the particles of sand are not brought into intimate contact. Free oil or an excess of asphalt in the binder, making it too rich, is liable to work up and be absorbed by the wearing surface, and thus cause it to disintegrate. The mixture may be chilled while being transported from the plant to the street. There

the plant to the street is great and there is any delay, some of the cement may work to the bottom of the load, and when it is dumped, there will be fat and lean spots, both of which are injurious. The paving mixture may be laid upon a damp or dirty foundation. There may be inadequate compression, for the importance of thorough compaction is not appreciated always and this portion of the work is slighted frequently.

(3) *Natural Causes.* All materials in nature continually are undergoing change due to the action of the elements, and to this asphalt is not an exception. Subjected to the action of heat, all bitumens grow harder, and when the maximum degree of hardness is attained, natural decay sets in so that under the combined action of the elements, the material gradually rots and disintegrates.

(4) *Defective Foundation.* By unequal settlement a weak foundation will cause cracks and depressions in the surface which will enlarge speedily under traffic. A porous foundation will permit the ground water to rise, by capillary action, to the underside of the wearing surface, where by freezing it will cause cracks and thus provide access for surface water; non-watertight connection between curbs and street fittings also furnishes a path for surface water to reach the underside of the wearing surface, where the presence of water causes rapid decay.

(5) *Other Causes.* Illuminating gas, escaping from leaking pipes under the pavement causes disintegration of the asphalt. Contraction, caused by the decrease in cementing power through aging of the asphalt, results in cracks. Due to an excess of fluxing material, there may be rolling and waving of the pavement under traffic. Injury may be caused by fires made upon the pavement, or by oil droppings from motor vehicles.

Sheet asphalt pavement usually is constructed under a contract that provides for its maintenance during a period of years (five or ten) by the contractor. Such a contract stipulates that the condition of the pavement at the expiration of the maintenance or guarantee period shall be as follows: Surface free from depressions exceeding $\frac{3}{4}$-inch deep, when measured between any two points 4 feet apart on a line conforming substantially to the original contour of the pavement. Free from cracks. Contain no disintegrated material. Thickness not reduced more than ¾ inch. Foundation free from cracks and settlement.

Rock Asphalt Pavement. This is the name applied to pavement made from the limestones and sandstones found naturally impregnated or cemented with bitumen. They are prepared for use by crushing and heating, and are used in their natural condition or mixed with other materials. Deposits are found in many parts of the United States and Europe. In Europe, rock asphalt is the material most extensively used for paving, under the name "asphalte". The European rock asphalts are impregnated very uniformly with from 7 per cent to 14 per cent of asphalt, and readily compact into a hard, smooth pavement which in frosty latitudes becomes very slippery. The American rock asphalts are impregnated irregularly with from 5 per cent to 30 per cent of asphalt. Their use for paving is limited, chiefly owing to the cost of transportation.

Asphalt Blocks. *Formation.* Asphalt paving blocks are formed from a mixture of asphaltic cement and crushed stone in the proportion of 8 or 12 per cent of cement to 88 or 92 per cent of stone. The materials are heated to a temperature of about 300° Fahrenheit, and mixed while hot in a suitable vessel. When the mixing is complete, the material is placed in molds and subjected to heavy pressure, after which the blocks are cooled suddenly by plunging into cold water. The usual dimensions of the blocks are 4 inches wide, 3 inches deep, and 12 inches long.

Foundation. The blocks usually are laid upon a concrete foundation with a cushion coat of sand about ½-inch thick. They are laid with their lengths at right angles to the axis of the street, and the longitudinal joints should be broken by a lap of at least 4 inches. The blocks then are rammed with hand hammers, or are rolled with a light steam roller, the surface being covered with clean, fine sand; no joint filling is used, as, under the action of the sun and traffic, the blocks soon become cemented.

The advantages claimed for a pavement of asphalt blocks over those for a continuous sheet of asphalt are: (1) that they can be made at a factory located near the materials, whence they can be transported to the place where they are to be used and can be machinery and skilled labor; (2) that they are less slippery, owing to the joints and the rougher surface due to the use of crushed stone.

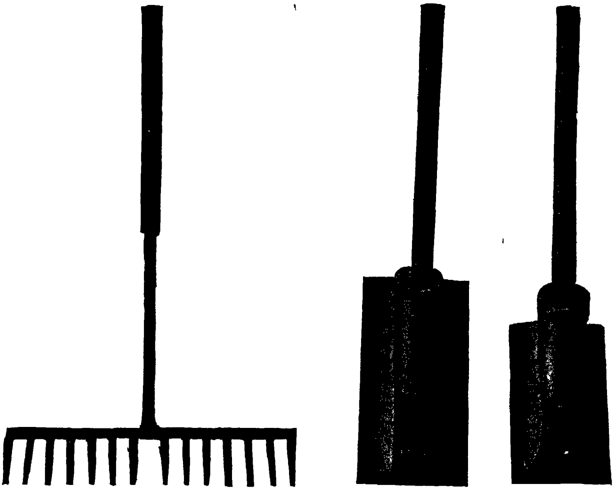

Fig 99. Rake and Smoothing Irons Used in Laying Asphalt Pavement
Courtesy of Barber Asphalt Paving Company, Philadelphia, Pennsylvania

Another Form. Another form of asphalt block, known as the "Lueba" block, consists of a block $8\frac{3}{4}$ inches long, $4\frac{1}{2}$ inches wide, and 4 inches thick, with the lower 3 inches composed of Portland-cement concrete covered with 1 inch of natural-rock asphalt; the two materials being joined under heavy hydraulic pressure. The blocks are laid on a concrete foundation and the joints between them are filled with hydraulic-cement grout.

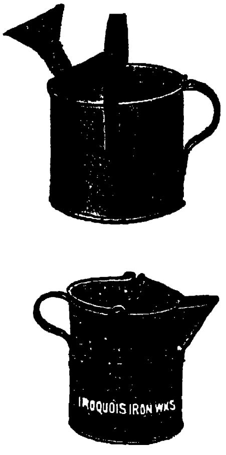

Fig. 100 Pouring Pots Used with Asphalt Pavements
Courtesy of Barber Asphalt

Tools Employed in Construction of Asphalt Pavements. The tools used in laying sheet-asphalt pavements comprise hand rammers iron, rakes, smoothing irons, Fig. 99; pouring pots, Fig. 100; hand rollers, either with or without a fire pot, Fig. 101; and steam rollers, with or without provision rollers are different in construction, appearance, and weight from those employed for compacting broken stone. The difference is due to the different character of the work required.

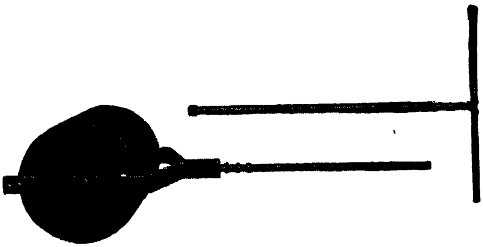

Fig 101. Hand Rollers Used in Laying Asphalt Pavements
Courtesy of Barber Asphalt Paving Company, Philadelphia, Pennsylvania

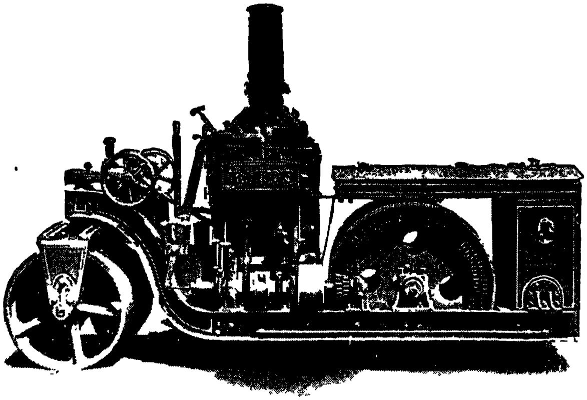

Fig 102 Small Road Roller Used in Laying Asphalt Pavements
Courtesy of Barber Asphalt Paving Company, Philadelphia, Pennsylvania

The principal dimensions of the 5-ton roller are as follows:

Front roll or steering wheel, diameter	30 to 32 inches
Rear roll or driving wheel, diameter	48 inches
Front roll, width .	40 inches
Rear roll, width.	40 inches
Length, extreme	14 feet
Height, extreme.	7 to 8 feet

MISCELLANEOUS PAVEMENTS

Under this head will be described briefly the most notable examples of pavements devised as substitutes for the recognized standard types, and sometimes used where good materials are not available, or where insufficient funds prevent their purchase, and in some cases for the purpose of utilizing waste products.

Burnt Clay. In the Mississippi Valley, during the dry season, the clay is cut from the roadway to a depth of about 2 feet, and piled so as to form enclosures about 15 feet in diameter and 2 feet high. After remaining so for about ten days to dry out, a fire is made in the inclosure, more dry clay placed on top and the burning proceeded with. The burnt clay after cooling, is relaid upon the road, and then, being of a thoroughly porous nature, settles into a dry, solid layer.

Straw. Clay roads have been improved by shaping and harrowing the road, then applying a layer of wheat straw, which is moistened with water, and cut and mixed with the clay by a disk harrow. More straw is added and the operation repeated, then compacted with a steam roller. The treatment is applied twice a year.

Oyster=Shell. The shells are spread on the road previously shaped and rolled. They crush readily and, possessing a high cementing quality, bind together to form a compact, smooth road surface, but owing to their softness, they are quickly ground to powder which is carried away readily by wind and rain water.

Chert. The siliceous material found overlying the red sandstone, which forms the covering of the red hematite iron ore in some of the Southern States, is used for both street and road paving. It is laid directly upon the earth surface or upon a prepared foundation, sprinkled, and compacted in the same manner as water-bound macadam.

Slag. The slag produced in the manufacture of iron and steel is used in various ways for paving. (1) It may be crushed to the desired sizes and used in the same manner as broken stone, laid in one or two courses, sprinkled and rolled. In some cases, a binder composed of quicklime is used; in others, a waste sulphite liquor is mixed with the water used for moistening it before rolling; and in others, it is mixed with coal-tar or other bituminous cement and formed into a pavement in the same manner as bituminous macadam. The pavement called "tarmac", a large amount of which has been used in England, is composed of slag, coal tar, rosin, and Portland cement. (2) The slag may be formed into blocks by casting in molds, which are used in the same manner as stone blocks. In this form they are called "scoria" or "slag" blocks.

Clinker. Where crematories are employed for the destruction of garbage about 33 per cent of the material remains after burning, in the form of clinker. This is broken up and ground to a fine powder, mixed with either a hydraulic or a bituminous cement, and pressed into blocks and slabs.

Petrolithic. Petrolithic paving is made by applying a bituminous oil to earth, sand, gravel, clay, or loam roads. The soil is plowed to a depth of at least 6 inches, pulverized by harrowing, and sprinkled with water. The bituminous oil is applied in one or two coats at the rate of 1 gallon per square yard, the oil and soil are mixed and compacted by a roller weighing 5000 pounds, the surface of which is studded with spikes having a flat head measuring 2×3 inches, and on which account it is named a "sheep's-foot" roller. In operation, the spikes or feet are forced into the loose soil and compress or pack it from the bottom upwards. After a thorough mixing and tamping, the surface is shaped with a road grader and rolled with a roller of the ordinary form.

Kleinpflaster. Kleinpflaster is the name given to a stone pavement used in Germany for exceptionally heavy traffic, and used also in England, under the name "durax". It is made of 3-inch cubes of hard stone, cut by machinery, and laid in small segments of circles. The stones are laid as close as possible and the joints are filled with hydraulic-cement grout or bituminous filler.

Iron. Several experiments have been made with iron for paving, but, while eminently durable, it was rough, noisy, and slippery, and its use either alone or combined with other materials has been abandoned.

Trackways. Formed of stone slabs, brick, concrete blocks, steel, and other materials, trackways have been constructed at various times for the purpose of reducing the resistance to traction. Their use on an extensive scale, however, has been abandoned

National Pavement. National pavement is composed of pulverized clay, loam, or ordinary soil, heated and mixed with liquid bitumen. The mixture is spread to a depth of 2 or 3 inches upon the surface of the compacted and drained natural soil and is compressed by a power roller.

Fibered Asphalt Pavement. Fibered asphalt pavement is composed of wood fiber, obtained as a waste product from the process of extracting tannin and asphalt. The fiber is heated and mixed with a predetermined quantity of asphalt. The hot mixture is run into molds forming small blocks which are shipped to the place of use. The blocks are there heated to a temperature of 275° F. in a traveling heater that moves along the roadway and from which the hot mixture emerges in a continuous stream 18 inches wide and is deposited on the previously prepared foundation to a depth of 4 inches. After spreading, it is compressed to a thickness of 2 inches with a power roller.

Westrumite. Westrumite is an asphalt cement temporarily liquefied by emulsification. It is mixed cold with broken stone in an ordinary concrete mixer, spread on the foundation, and compacted by rolling. The evaporation of the vehicle leaves the asphalt cement as the binder.

MISCELLANEOUS STREET WORK

FOOTPATHS

A footpath or walk is simply a road under another name—a road for pedestrians instead of one for horses and vehicles. The only difference that exists is in the degree of service required; but the conditions of construction that render a road well adapted to its object are very much the same as those required for a walk.

The effects of heavy loads such as traverse carriage-ways are not felt upon footpaths; but the destructive action of water and frost is the same in either case, and the treatment to counteract or resist these elements as far as practicable, and to produce permanency, must be the controlling idea in each case, and should be carried out upon a common principle. It is not less essential that a walk should be well adapted to its object than that a road should be; and it is annoying to find it impassable or insecure and in want of repair when it is needed for convenience or pleasure. In point of economy, there is the same advantage in constructing a footway skilfully and durably as there is in the case of a road.

Width. The width of footwalks (exclusive of the space occupied by projections and shade trees) should be ample to accommodate comfortably the number of people using them. In streets devoted entirely to commercial purposes, the clear width should be at least one-third the width of the carriage-way; in residential and suburban streets, a very pleasing result can be obtained by making the walk one-half the width of the roadway, and by devoting the greater part to grass and shade trees.

Cross Slope. The surface of footpaths must be sloped so that the surface water will readily flow to the gutters. This slope need not be very great; $\frac{1}{8}$ inch per foot will be sufficient. A greater slope with a thin coating of ice upon it, becomes dangerous to pedestrians.

Foundation. As in the case of roadways, so with footpaths, the foundation is of primary importance. Whatever material may be used for the surface, if the foundation is weak and yielding, the surface will settle irregularly and become extremely objectionable, if not dangerous, to pedestrians.

Surface. The requirements of a good covering for sidewalks are:

(1) It must be smooth but not slippery.

(2) It must absorb the minimum amount of water, so that it may dry rapidly after rain.

(3) It must not be abrased easily.

(4) It must be of uniform quality throughout, so that it may wear evenly.

(5) It must neither scale nor flake.

(6) Its texture must be such that dust will not adhere to it.

(7) It must be durable.

Materials. The materials used for footpaths are as follows: stone, natural and artificial; wood; asphalt; brick; tar concrete; and gravel.

Stones. Of the natural stones, sandstone (bluestone) and granite are employed extensively. The bluestone, when well laid, forms an excellent paving material. It is of compact texture, absorbs water to a very limited extent, and hence soon dries after without becoming excessively slippery. Granite, although exceedingly durable, wears very slippery, and its surface has to be roughened frequently.

Slabs, of whatever stone, must be of equal thickness throughout their entire area; the edges must be dressed true to the square for the whole thickness (edges must not be left feathered as shown in Fig. 103); and the slabs must be bedded solidly on the foundation and the joints filled with cement mortar. Badly set or faultily dressed flagstones are very unpleasant to walk over, especially in rainy weather; the unevenness causes pedestrians to stumble, rocking stones squirt dirty water over their clothes.

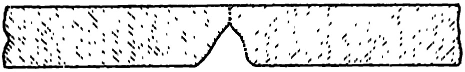

Fig. 103. Faulty Joint in Stone Sidewalk

Wood. Wood has been used largely in the form of planks; it is cheap in first cost, but proves very expensive from the fact that it lasts but a comparatively short time and requires constant repair to keep it from becoming dangerous.

Asphalt. Asphalt forms an excellent footway pavement; it is durable and does not wear slippery.

Brick. Brick of suitable quality, well and carefully laid on a concrete foundation, makes an excellent footway pavement for residential and suburban streets of large cities, and also for the main streets of smaller towns. The bricks should be good qualities of paving brick (ordinary building brick are unsuitable, as they soon wear out and are broken easily). The bricks should be laid in parallel rows on their edges, with their lengths at right angles to the axis of the path.

Concrete. Concrete or artificial stone is used extensively as a footway paving material. Its manufacture is the subject of several patents, and numerous kinds are to be had in the market. When manufactured of first-class materials and laid in a substantial manner, with proper provision against the action of frost, artificial stone forms a durable, agreeable, and inexpensive pavement.

Concrete walks are formed in one or two courses. In one-course work, the concrete is laid to a depth of 4 inches and tamped until sufficient mortar flushes to the surface to permit the forming of a smooth surface. In two-course work, the concrete for the base is spread and tamped to a depth of 3 inches, the top or surface course is spread upon the base before the latter has begun to set. The top course has a thickness of about 1 inch, and it is tamped and its surface is brought to the required plane by a straightedge and by troweling. Expansion is provided for by transverse joints extending the full depth of the concrete. The joints are placed 4 feet apart and are formed by placing across the side forms a $\frac{1}{4}$-inch thick metal dividing strip, which is removed before the cement sets so that the edges of the joint may be smoothed and rounded with a suitable tool.

The area to be covered by the walk is excavated to a minimum depth of 8 inches, or to such greater depth as the nature of the ground may require to secure a solid foundation. The surface of the ground so exposed is compacted by ramming, and a drainage course is formed of broken stone, gravel, or steam-plant cinders, thoroughly compacted by ramming, and its surface is brought to a plane parallel to and 4 inches below the finished surface of the concrete. In some situations it may be necessary to connect the drainage course with the sewers, street drains, or side ditches, for the purpose of furnishing an outlet for standing water; this is done by the use of 3-inch drain pipe placed where required.

The forms of steel or wood should be made substantially, and left in place until the concrete is set hard.

Concrete walks fail from the use of improper materials and defective workmanship, insufficient expansion joints, heaving and cracking by frost, due to imperfect drainage, displacement and cracks, due to settlement of the drainage course—this latter being frequent when cinders are used, as in time they are liable to decompose and shrink in volume and thus allow the concrete to settle. In two-course work failure may be in respect to flaking of the surface by the action of water and frost entering between and separating the courses. The concrete should not be laid during freezing weather. nor should frozen materials be used

CURBSTONES AND GUTTERS

Curbstones. Curbstones are employed for the outer side of footways, to sustain the covering and to form the gutter. Their upper edges are set flush with the footwalk pavement, so that the water can flow over them into the gutters.

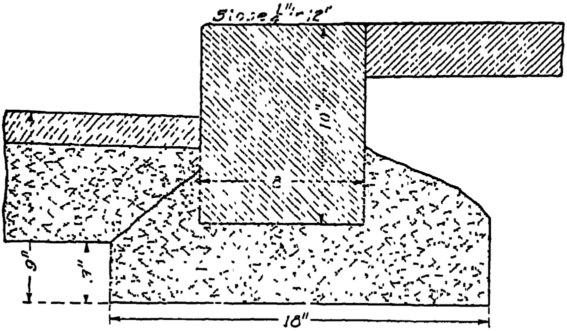

Fig. 104. Typical Section Showing Stone Curb Eight Inches Thick

The disturbing forces which the curb has to resist are: (1) The pressure of the earth behind it, which is frequently augmented by piles of merchandise, building materials, etc. This pressure tends to overturn it, break it transversely, or move it bodily on its base. (2) The pressure due to the expansion of freezing earth behind and beneath the curb. This force is most frequent where the sidewalk is sodded partly and the ground accordingly is moist. Successive freezing and thawing of the earth behind the curb will occasion a succession of thrusts forward, which, if the curb be of faulty

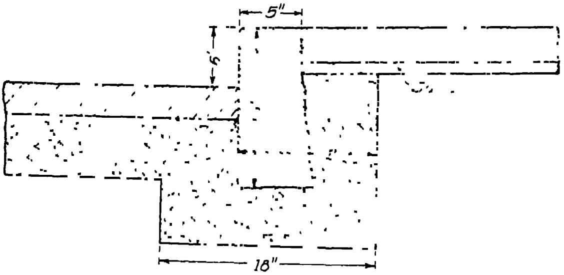

Fig 105 Typical Section Showing Stone Curb Five Inches Thick

(3) The concussions and abrasions caused by traffic. To withstand the destructive effect of wheels, curbs are faced with iron; and a concrete curb with a rounded edge of steel has been patented and used to some extent. Fires built in the gutters deface and seriously injure the curb. Posts and trees set too near the curb, tend to break, displace, and destroy it.

The use of drain tiles under the curb is a subject of much difference of opinion among engineers. Where the subsoil contains water naturally, or is likely to receive it from outside the curb lines, the use of drains is of decided benefit; but great care must be exercised in jointing the draintiles, lest the soil shall be loosened and removed, causing the curb to drop out of alignment.

The materials employed for curbing are the natural stones—as granite, sandstone (bluestone), etc.; artificial stone—fire clay, and cast iron.

The dimensions of curbstones vary considerably in different localities and according to the width of the footpaths; the wider the path, the wider should be the curb. However, it should be never less than 8 inches deep, nor narrower than 4 inches. Depth is necessary to prevent the curb's turning over toward the gutter. It never should be in smaller lengths than 3 feet. The top surface should be beveled off to conform to the slope of the footpath. The front face should be hammer-dressed for a depth of about 6 inches, in order that there may be a smooth surface visible against the gutter. The back for 3 inches from the top also should be dressed, so that the flagging or other paving may butt fair against it. The end joints should be cut a true square the full thickness of the stone at the top, and so much below the top as will be exposed; the remaining portion of the depth and bottom should be squared roughly, and the bottom should be fairly parallel to the top. (See Figs. 104 and 105.)

Combination Curb and Gutter. Concrete curb and gutter combined is constructed by placing the concrete in suitable forms. The concrete should be handled so as to prevent the separation of the stone and mortar, and when placed should be tamped well to bring the mortar to the surface and make complete contact with the forms. The corner formed by the top and face surfaces is formed of a steel bar put in place before the concrete is laid and anchored by metal strips spaced about 3 feet apart. Expansion joints are formed at distances of 10 or 12 feet. The remarks made under concrete walks regarding foundation, drainage, failure, etc., apply also to concrete curbs.

STREET CLEANING

The cleaning of streets is practiced for the purpose of protecting the health of the neighboring residents and for the comfort of the users. It is of comparatively recent development, and is rendered possible only by the use of hard pavements. The materials to be removed from the streets consist of animal droppings, material worn from the pavement, materials dropped from vehicles, waste from building construction, miscellaneous materials swept from houses, stores, and factories, and the accumulation of snow during winter.

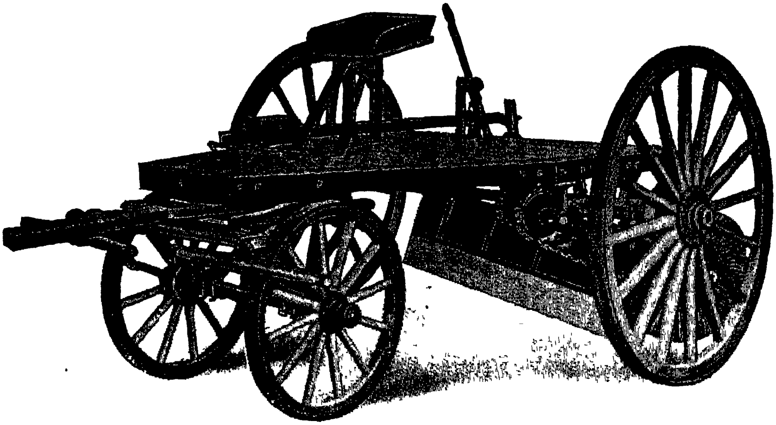

Fig 106. Typical Machine Street Sweeper
Courtesy of Acme Road Machinery Company, Frankfort, New York

Cleaning Methods. The local conditions and character of the traffic and pavement determine the methods to be employed and the intervals for cleaning the streets. The methods employed are: sweeping, either by hand or by machine brooms; and flushing with water—the work being performed either during the day or the night, by large gangs at night, and by means of a patrol system during the day. Fig. 106 shows one of the machine sweepers used.

TABLE XV

Rate and Cost of Street Cleaning

Pavement	Approximate Surface Swept per Man (sq. yds. per hr)		Approximate Dirt from Daily Sweeping (cu. yds. per 1000 sq. yds)		Average Cost per Each Cleaning (cents per sq. yd.)
	(Wet)	(Dry)	(Min.)	(Max.)	
Asphalt	1000	1200	.007	.040	.0030
Granite-block	750	1000	.015	.024	.0050
Macadam (water-bound)	700		.100	.350	.0106
Wood			.070	.200	0070
Brick					.0034

In the hand-cleaning method by day patrol, each man is furnished with a push broom, shovel, and can carrier in which to place the refuse, and has a certain section of street to clean each day. The day patrol sometimes is supplemented by a large gang working during the night. When machine brooms are employed they usually are operated at night and are supplemented by the patrol system during the day. As to which is the most economical, it will depend upon the cost of labor and the condition of the pavements; on pavements covered with ruts and depressions machine brooms are ineffective.

The approximate costs of the various methods of street cleaning per 1000 sq. yds. are:

Sweeping (hand)	$0 281
Sweeping (machine)	0 317
Flushing (hand-hose)	0 319
Flushing (machine)	0 721

The average cost of supervision varies from .011 cent to 34 cents per mile.

The amount of surface cleaned by a machine broom depends upon the width of the broom, the power of the horses or other motive power, gradient, and condition of the surface. The wider the broom the less will be the cost. The average speed of travel is about 1½ miles per hour.

In Table XV are indicated the amount of surface which an average man will sweep per hour, depending upon the condition

Fig 107 Snow Roller for Compacting Snow in Streets and Walks
Courtesy of Acme Road Machinery Company, Frankfort, New York

produced by the different pavements, if swept daily; and the average cost of cleaning different pavements.

Removal of Snow. The methods employed for keeping roads and streets passable during the period of snowfall varies according to the climatic conditions. In localities subject to heavy falls of snow, and continuous low temperature that retards the melting of the snow until spring, two methods are followed: (1) a narrow track is opened by a snowplow, through the center of the road, the snow being formed into long, narrow heaps on each side; (2) the snow is not removed, but is compacted by rolling with a light-weight wood or metal roller, Fig. 107. In localities having light falls and in the larger cities, the snow is pushed by plows or rotary brooms toward the gutters from where it is loaded into vehicles, hauled to a natural waterway and dumped, or in the absence of this it is placed in vacant lots and in some cases it is disposed of by dumping into the sewers through the manholes, but this must be done carefully, as there is liability of choking the sewer by the snow's consolidating. Light falls may be disposed of by the application of a stream of water to the surface of the street thereby washing the snow into the sewer. Many machines have been devised for melting the snow by the application of steam, hot air, etc., but none of them have been successful economically. In some cities the snow is melted by an application of rock salt which produces a thawing action when mixed with the snow by the traffic, the slushy mixture so formed is swept to the gutters by machine brooms and washed into the sewers by a stream of water from the hydrants. Objection is made to this method on account of the intense cold produced and its injurious effect upon the feet of pedestrians and on the hoofs of horses.

In order to cause the minimum of inconvenience to traffic it is necessary that the snow be removed from the streets as quickly as possible, therefore, it is customary, before the arrival of winter, to lay out the method and organization required and to make arrangements for the quick mobilization of the force needed for its removal. To accomplish this the city is divided into districts, in each of which there is established a headquarters and depot stocked with the necessary tools to execute the work in that district, and to which

Street Sprinkling. Streets and roads are sprinkled with water for the purpose of abating dust and cooling the air. While water-bound macadam and earth surfaces must be sprinkled to abate the dust, a stone-block, brick, asphalt, or wood pavement will not require sprinkling if thoroughly cleansed and kept clean. On unclean and badly maintained pavements, sprinkling with water as usually performed converts the fine dust into a slime which renders all smooth pavements slippery, and in warm weather it becomes a prolific breeding place for disease germs, it clings to the feet and clothing of pedestrians, and, with its accompanying germs, is carried into buildings and dwellings.

The average cost of sprinkling per square yard is $0.009.

The systems followed for executing the work of street cleaning, snow removal, and sprinkling are: (1) by contract where the contractor furnishes all the tools and labor; (2) by contract for the labor only, the city furnishing the tools and machinery; (3) by the city, with its own staff and machinery.

SELECTING THE PAVEMENT

The problem of selecting the best pavement for any particular case is a local one, not only for each city, but also for each of the various parts into which the city is imperceptibly divided; and it involves so many elements that the nicest balancing of the relative values for each kind of pavement is required in arriving at a correct conclusion.

In some localities, the proximity of one or more paving materials determines the character of the pavement; while in other cases a careful investigation may be required in order to select the most suitable material. Local conditions always should be considered; hence it is not possible to lay down any fixed rule as to what material makes the best pavement.

Qualifications. The qualities essential to a good pavement may be stated as follows:

(1) It should be impervious.

(2) It should afford good foothold for horses and adhesion for motor vehicles.

(3) It should be hard and durable, so as to resist wear and disintegration.

(4) It should be adapted to every grade.

(5) It should suit every class of traffic.
(6) It should offer the minimum resistance to traction.
(7) It should be noiseless.
(8) It should yield neither dust nor mud.
(9) It should be cleaned easily.
(10) It should be cheap.

Interests Affected. Of the above requirements, numbers (2), (4), (5), and (6) affect the traffic and determine the cost of haulage by the limitations of loads, speed, and wear and tear of horses and vehicles. If the surface is tough or the foothold bad, the weight of the load a horse can draw is decreased, thus necessitating the making of more trips or the employment of more horses and vehicles to move a given weight. A defective surface necessitates a reduction in the speed of movement and a consequent loss of time; it increases the wear of horses, thus decreasing their life service and lessening the value of their current services; it also increases the cost of maintaining vehicles and harness.

Requirements, numbers (7), (8), and (9), affect the occupiers of adjacent premises, who suffer from the effect of dust and noise; they also affect the owners of said premises, whose income from rents is diminished where these disadvantages exist. Numbers (3) and (10) affect the taxpayers alone—first, as to the length of time during which the covering remains serviceable; and second, as to the amount of the annual repairs. Number (1) affects the adjacent occupiers principally on hygienic grounds. Numbers (7) and (8) affect both traffic and occupiers.

Problem Involved in Selection. The problem involved in the selection of the most suitable pavement consists of the following factors: (1) adaptability; (2) desirability; (3) serviceability; (4) comparative safety; (5) durability; (6) cost.

Adaptability. The best pavement for any given roadway will depend altogether on local circumstances. Pavements must be adapted to the class of traffic that will use them. The pavement suitable for a road through an agricultural district will not be suitable for the streets of a manufacturing center; nor will the covering suitable for heavy traffic be suitable for a pleasure drive or for a residential district.

General experience indicates the relative fitness of the several

TABLE XVI

Resistance to Traction on Different Pavements

Kind of Pavement	Tractive Resistance	
	Lb per Ton	Fraction of the Load
Sheet-asphalt	30 to 70	$\frac{1}{67}$ to $\frac{1}{30}$
Brick	15 to 40	$\frac{1}{133}$ to $\frac{1}{50}$
Cobblestone	50 to 100	$\frac{1}{40}$ to $\frac{1}{20}$
Stone-block	30 to 80	$\frac{1}{67}$ to $\frac{1}{25}$
Rectangular wood-block	30 to 50	$\frac{1}{67}$ to $\frac{1}{40}$
Round wood-block	40 to 80	$\frac{1}{50}$ to $\frac{1}{25}$

materials as follows: for country roads, suburban streets, and pleasure drives—broken stone; for streets having heavy and constant traffic —rectangular blocks of stone, laid on a concrete foundation, with the joints filled with bituminous or Portland-cement grout; for streets devoted to retail trade, and where comparative noiselessness is essential—asphalt, wood, or brick.[4] More recent experience indicates that concrete, when properly laid and reinforced at necessary points, may be employed to advantage for any pavement, both as base and as wearing surface.

Desirability. The desirability of a pavement is its possession of qualities which make it satisfactory to the people using and seeing it. Between two pavements alike in cost and durability, people will have preferences arising from the condition of their health, personal prejudices, and various other intangible influences, causing them to select one rather than the other in their respective streets. Such selections often are made against the demonstrated economies of the case, and usually in ignorance of them. Whenever one kind of pavement is more economical and satisfactory to use than is any other, there should not be any difference of opinion about securing it, either as a new pavement or in the replacement of an old one.

The economic desirability of pavements is governed by the ease of movement over them, and is measured by the number of horses or pounds of tractive force required to move over them a given weight—usually 1 ton. The resistance offered to traction by different pavements is shown in Table XVI.

Serviceability. The serviceability of a pavement is its quality of fitness for use. This quality is measured by the expense caused to the traffic using it—namely, the wear and tear of horses and vehicles, loss of time, etc. No statistics are available from which to deduce the actual cost of wear and tear.

The serviceability of any pavement in great measure depends upon the amount of foothold afforded by it to the horses—provided, however, that its surface be not so rough as to absorb too large a percentage of the tractive energy required to move a given load over it. Cobblestones afford excellent foothold, and for this reason are largely employed by horse-car companies for paving between the rails; but the resistance of their surface to motion requires the expenditure of about 40 pounds of tractive energy to move a load of 1 ton. Asphalt affords the least foothold; but the tractive force required to overcome the resistance it offers to motion is only about 30 pounds per ton.

Comparative Safety. The comparison of pavements in respect to safety, is the average distance traveled before a horse falls. The materials affording the best foothold for horses are as follows, stated in the order of their merit:

(1) Earth, dry and compact.
(2) Gravel.
(3) Broken stone (macadam).
(4) Wood.
(5) Sandstone and brick.
(6) Asphalt.
(7) Granite blocks.

Durability. The durability of pavement is that quality which determines the length of time during which it is serviceable, and does not relate to the length of time it has been down. The only measure of durability of a pavement is the amount of traffic tonnage it will bear before it becomes so worn that the cost of replacing it is less than the expense incurred by its use.

As a pavement is a construction, it necessarily follows that there is a vast difference between the durability of the pavement and the durability of the materials of which it is made. Iron is eminently durable; but, as a paving material, it is a failure.

The durability of a paving material will vary considerably with

TABLE XVII

Terms of Life of Various Pavements

MATERIALS	TERMS OF LIFE (Years)
Granite-block	12 to 30
Sandstone	6 to 12
Asphalt	10 to 14
Wood	7 to 15
Limestone	1 to 3
Brick	5 to 15
Macadam	5 to ?

the condition of cleanliness observed. One inch of overlying dirt will protect the pavement most effectually from abrasion, and prolong its life indefinitely. But the dirt is expensive; it injures apparel and merchandise, and is the cause of sickness and discomfort. In the comparison of different pavements, no traffic should be credited to the dirty one. The life or durability of different pavements under like conditions of traffic and maintenance, may be taken as shown in Table XVII.

Cost. First cost or the cost of construction, is largely controlled by the locality of the place, its proximity to the particular material used, and the character of the foundation. The question of cost is the one which usually interests taxpayers, and is problably the greatest stumblingblock in the attainment of good roadways. The first cost usually is charged against the property abutting on the highway to be improved. The result is that the average property owner always is anxious for a pavement that costs little, because he must pay for it, not caring for the fact that cheap pavements soon wear out and become a source of endless annoyance and additional expense. Thus false ideas of economy usually have stood, and undoubtedly always will stand to some extent, in the way of realizing that the best is the cheapest.

The pavement which has cost the most is not always the best; nor is that which cost the least the cheapest; the one which is truly the cheapest is the one which makes the most profitable returns in proportion to the amount expended upon it. No doubt there is a limit of cost to go beyond which would produce no practical benefit; but it always will be found more economical to spend enough to secure the best results, and this always will cost less in the long run. One dollar well spent is many times more effective than one-half of the amount injudiciously expended in the hopeless effort to reach sufficiently good results. The cheaper work may look as well as the more expensive, for a time, but very soon may have to be done over again.

Economic Benefit. The economic benefit of a good roadway is comprised in the following: its cheaper maintenance, the greater facility it offers for traveling, thus reducing the cost of transportation; the lower cost of repairs to vehicles, and less wear of horses, thus increasing their term of serviceability and enhancing the value of their present service; the saving of time; and the ease and comfort afforded to those using the roadway.

Relative Economies. The relative economies of pavements—whether of the same kind in different condition, or of different kinds in like good condition—are determined sufficiently by summing their cost under the following headings of account:

(1) Annual interest upon first cost and sinking fund.
(2) Annual expense for maintenance.
(3) Annual cost for cleaning and sprinkling.
(4) Annual cost for service and use.
(5) Annual cost for consequential damages.

Interest on First Cost. The first cost of a pavement, like any other permanent investment, is measurable for purposes of comparison by the amount of annual interest on the sum expended. Thus, assuming the worth of money to be 4 per cent, a pavement costing $4 per square yard entails an annual interest loss or tax of $0.16 per square yard.

Cost of Maintenance. Under this head must be included all outlays for repairs and renewals which are made from the time when the pavement is new and at its best to a time subsequent, when, by any treatment, it is put again in equally good condition. The gross sum so derived, divided by the number of years which elapse between the two dates, gives an annual average cost for maintenance.

Maintenance means the keeping of the pavement in a condition practically as good as when first laid. The cost will vary conwhich it is constructed, but upon the condition of cleanliness observed, and the quantity and quality of the traffic using the pavement.

The prevailing opinion that no pavement is a good one unless, when once laid, it will take care of itself, is erroneous; there is no such pavement. All pavements are being worn constantly by traffic and by the action of the atmosphere; and if any defects which appear are not repaired quickly, the pavements soon become unsatisfactory and are destroyed. To keep them in good repair, incessant attention is necessary, and is consistent with economy. Yet claims are made that particular pavements cost little or nothing for repairs, simply because repairs in these cases are not made, while any one can see the need of them.

Cost of Cleaning and Sprinkling. Any pavement, to be considered as properly cared for, must be kept dustless and clean. While circumstances legitimately determine in many cases that streets must be cleaned at daily, weekly, or semiweekly intervals, the only admissible condition for the purpose of analysis of street expenses must be that of like requirements in both or all cases subjected to comparison.

The cleaning of pavements, as regards both efficiency and cost, depends (1) upon the character of the surface; (2) upon the nature of the materials of which the pavements are composed. Block pavements present the greatest difficulty; the joints can never be perfectly cleaned. The order of merit as regards facility of cleansing, is: (1) asphalt, (2) concrete, (3) brick, (4) stone, (5) wood, (6) macadam.

Cost of Service and Use. The annual cost for service is made up by combining several items of cost incidental to the use of the pavement for traffic—for instance, the limitation of the speed of movement, as in cases where a bad pavement causes slow driving and consequent loss of time; or cases where the condition of a pavement limits the weight of the load which a horse can haul, and so compels the making of more trips or the employment of more horses and vehicles; or cases where conditions are such as to cause greater wear and tear of vehicles, of equipment, and of horses. If a vehicle is run 1500 miles in a year, and its maintenance cost $30 a year, then the cost of its maintenance per mile traveled is 2 cents. If the value of a team's time is, say $1 for the legitimate time taken in going 1 mile with a load, and in consequence of bad roads it takes double that time, then the cost to traffic from having to use that mile of bad roadway is $1 for each load. The same reasoning applies to circumstances where the weight of the load has to be reduced so as to necessitate the making of more than one trip. Again, bad pavements lessen not only the life service of horses, but also the value of their current service.

Cost for Consequential Damages. The determination of consequential damages arising from the use of defective or unsuitable pavements, involves the consideration of a wide array of diverse circumstances. Rough-surfaced pavements, when in their best condition, afford a lodgment for organic matter composed largely of the urine and excrement of the animals employed upon the roadway. In warm and damp weather, these matters undergo putrefactive fermentation, and become the most efficient agency for generating and disseminating noxious vapors and disease germs, now recognized as the cause of a large part of the ills afflicting mankind. Pavements formed of porous materials are objectionable on the same, if not even stronger, grounds.

Pavements productive of dust and mud are objectionable, and especially so on streets devoted to retail trade. If this particular disadvantage be appraised at so small a sum per lineal foot of frontage as $1.50 per month, or 6 cents per day, it exceeds the cost of the best quality of pavement free from these disadvantages.

Rough-surfaced pavements are noisy under traffic and insufferable to nervous invalids, and much nervous sickness is attributable to them. To all persons interested in nervous invalids, this damage from noisy pavements is rated as being far greater than would be the cost of substituting the best quality of noiseless pavement; but there are, under many circumstances, specific financial losses, measurable in dollars and cents, dependent upon the use of rough, noisy pavements. They reduce the rental value of buildings and offices situated upon streets so paved—offices devoted to pursuits wherein exhausting brain work is required. In such locations, quietness is almost indispensable, and no question about the cost

TABLE XVIII

Comparative Rank of Pavements

CHARACTERISTICS		VARIETY								
Qualities	Value (per cent)	Asphalt (sheet)	Asphalt (block)	Concrete	Macadam (bituminous)	Macadam (water-bound)	Brick	Granite	Sandstone	Wood
Low tractive resistance	20	20.0	19.0	18.0	19.0	11.0	18.0	12.0	14.0	20.0
Service on grades	10	3.0	3.0	7.0	4.0	8.0	9.0	10.0	10.0	2.0
Non-slipperiness	5	1.5	2.5	4.0	2.5	4.5	3.5	3.5	5.0	2.0
Favorableness to travel	5	5.0	4.5	3.5	4.0	4.5	3.5	3.5	4.0	4.5
Sanitariness	10	10.0	9.0	7.0	8.0	3.0	8.0	6.0	7 0	9.0
Noiselessness	3	2.5	2.5	2.0	2.5	2.5	1.5	1.0	1.5	3.0
Minimum dust	3	2.5	2.5	2.0	2.0	1.0	2.0	1.5	2 0	2.0
Ease of cleaning	5	5.0	5.0	3.5	4.0	1.0	3.5	1.5	1.5	5.0
Acceptability	4	3.5	3.5	2.5	3.0	1.5	2.5	2.0	2.5	4.0
Durability	15	7.5	8.5	6.0	3.0	1.5	10.0	15.0	14.0	11.5
Ease of maintenance	5	3.5	4.0	3.0	3.0	2.5	4.0	4.5	5.0	5.0
Cheapness (first cost)	10	4.5	4.0	5.0	7.5	10.0	4.0	3.0	3.5	3.0
Low annual cost	5	1.5	2.5	3.0	3.5	1.0	4.5	5.0	5.0	5.0
Totals....	100	70 0	70.5	66.5	66.0	52.0	74.0	68.5	75.0	76.0
Approximate first cost (dollars per sq. yd.)..		2 30	2.65	1.85	1.35	1.00	2.65	3.25	3.00	3.45

When an investigator has done the best he can to determine such a summary of costs of a pavement, he may divide the amount of annual tonnage of the street traffic by the amount of annual costs, and know what number of tons of traffic are borne for each cent of the average annual cost, which is the crucial test for any comparison, as follows:

(1) Annual interest upon first cost and sinking fund. . . $
(2) Average annual expense for maintenance and renewal .
(3) Annual cost for custody (sprinkling and cleaning)......
(4) Annual cost for service and use..... . .
(5) Annual cost for consequential damages.... . ..
Amount of average annual cost. ....
Annual tonnage of traffic . . . .
Tons of traffic for each cent of cost.. ....

Gross Cost of Pavements. Since the cost of a pavement roadway, the extent and nature of the traffic, and the condition of repair and cleanliness in which it is maintained, it follows that in no two streets is the endurance or the cost the same, and the difference between the highest and lowest periods of endurance and amount of cost is very considerable.

Comparative Rank of Pavements. In Table XVIII is given the rank of the various pavements in percentage, prorated from the values assigned in the first column to the desired qualities. The pavement ranking first in any given quality is given the full value for that quality, the others grading down from this value according to the extent to which they possess the desired quality. An examination of the table shows macadam to be the cheapest; least durable, and most difficult to maintain and cleanse; rather favorable to travel; comparatively low in sanitariness; and high in annual cost. While the table may be used as an aid in determining the most suitable pavement according to the factors that are susceptible of a numerical value, the values assigned must be modified by local conditions; first cost will necessarily vary in different localities, and certain factors will be more important in one locality than another.

Specifications. A specification or detailed description of the various works to be carried out always is attached to a contract, and is prepared before estimates are called for. The prominent points that are essential to the production of a specification that will fulfill its purpose properly, are: (1) description of the work; (2) extent of the work; (3) quality of the materials; (4) tests for the materials; (5) delivery of the materials; (6) character of the workmanship; (7) manner of executing the work.

Attention to these points and a clear and accurate description of each detail (leaving nothing to be imagined) not only will contribute materially to the rapid and efficient execution of the work, but will avoid any future misunderstanding. Every item of the work should be allotted a separate clause, for confusion must ensue when a single clause includes descriptions of several matters.

As a rule it is undesirable to insert in specifications any dimensions or weights that can be shown on the drawings. However, of numerals; the use of numerals, and particularly decimal numbers, should be avoided, as there is a risk of having them set up incorrectly by the typesetter and overlooked in the proofreading. When a numeral is used it should be followed by the word or words indicating the numeral, placing the numeral in parenthesis.

Brevity, so far as it is consistent with completeness, should prevail, but the word "et cetera" should be excluded rigidly, and the matters covered by it should be defined clearly. Neither should important points of the work be dismissed with the direction that "the work shall be done to the satisfaction of the engineer". A direction of this kind usually implies that the engineer does not know what he wants, and therefore leaves the matter to the superior knowledge of the contractor—an attitude not very creditable to the former. The only really legitimate use of this phrase is in a general clause referring to the whole of the work.

The specifying of impracticable sizes of materials must be avoided as it causes unnecessary discussion and frequently leads to a charge for "extras".

A clause or phrase permitting the furnishing of alternative materials or workmanship should be excluded, because such permission affords ground for dispute and difference of opinion. On the other hand, specifying that certain articles manufactured by a particular firm shall be used should be avoided, as it suggests unfairness on the part of the engineer, and may create the idea that his selection is not without profit to himself.

With regard to the actual methods of carrying out the work, the contractor should not be tied to any particular means of effecting the required end, unless special circumstances require it, for, provided the materials and workmanship are satisfactory, it is better to allow the contractor to use his own discretion as to the manner of producing the required result.

While the standard and proper tests for the materials always should be stipulated, yet if they are carried to an extreme degree, as frequently happens, they defeat their own object. When it becomes impossible to carry out certain unreasonable demands, the alternative is to evade them as much as possible; and it must be borne in mind that the more stringent the demand, the greater

Contracts. A good, clear, and comprehensive contract is a difficult thing to write, but it should be "common sense" from beginning to end, and should be the joint production of both engineering and legal ability, neither sacrificing the one feature to the other.

The stipulations of the contract form the legal part of the document and are distinct from the technical description of the work to be done. The essential points are: (1) time of commencement; (2) time of completion; (3) manner and times of payment; (4) prices for which the work is to be performed; (5) measurements; (6) damages for noncompletion; (7) protection of persons and property during the prosecution of work; (8) such special stipulations as may be required for the particular work that is being contracted for.

It should be borne in mind that the contract and specifications when duly signed by the parties interested, is a legal document, which must be produced in court in the event of a dispute arising, therefore, it is of the utmost importance that it be written clearly in simple language, the clauses being arranged in logical sequence, and the descriptions made exact and complete without being needlessly verbose.

High-sounding phrases, and duplication of statements or information, should be avoided as tending to confusion. Specifications are seldom judged by literary standards of excellence, therefore, words may be repeated again and again if they express the meaning of the writer more clearly and forcibly than an alternative would do.

In the case of a lengthy contract and specification, a complete index with the clause and page numbers will be found an aid to finding quickly any required subject; cross references may sometimes be introduced with advantage.

REVIEW QUESTIONS

ON THE SUBJECT OF

PLANE SURVEYING

PART I

1. Explain by a diagram how to erect (with the tape alone) a line at right angles to a given line.

2. The sides of a triangular field are 826,432 and 529 feet. Find the area of the field in acres, roods, and square rods.

3. Find the area of a triangle whose sides are 31, 40, and 55 rods.

4. Given in Fig. 14, $CB = 2.85$ chains, $CD = 3.67$ chains, $CS = CL = 0.52$ chains, and $LS = 0.75$ chains. Calculate the area of the triangle BCD.

5. A certain line is known to be 530 feet in length, but when measured with a certain tape is found to be $533\frac{1}{2}$ feet in length. Determine the true length of the tape.

6. A certain field is measured with a Gunter's chain and is found to contain 5.75 acres. It is afterwards discovered that the chain is $\frac{3}{10}$ of a foot too long. Find the true area of the field.

7. If a line as measured is found to be $432\frac{3}{4}$ feet in length and it is afterwards discovered that the tape is too short by $\frac{1}{8}$ of a foot, what is the true length of the line?

8. A level bubble has a radius of 150 feet and its scale has 10 spaces in an inch. What error in leveling will result at a distance of 275 feet when the level bubble is $1\frac{1}{2}$ spaces out of level?

9. At a distance of 150 feet, two rod readings were 3.704 and 3.745 and the bubble moved over $\frac{1}{8}$ inch. Determine the radius of the bubble tube.

10. What error in leveling will result at a distance of 123 feet if the bubble is $2\frac{3}{4}$ spaces out of level, the scale of which has 7 spaces in an inch, the radius being 176 feet?

REVIEW QUESTIONS

ON THE SUBJECT OF

PLANE SURVEYING

PART II

---

1. Give a detailed description of the transit.

2. Give the *reasons* for each of the adjustments of the transit. Draw a diagram in each case.

3. Describe fully *in their order* how to test the adjustments of the transit.

4. Describe fully in their order how to make the several adjustments of the transit.

5. Define the following: Latitude of a station; longitude of a line; double longitude; departure.

6. Explain the use of double longitudes. Draw a figure, and explain how latitudes and departures may be gotten from the length and bearing of a line.

7. In order to find the difference in height of two peaks M and N, a base line AB was laid off 5,000 feet long; and the horizontal angles $BAM = 120^\circ 30'$, $BAN = 49^\circ 15'$, $ABM = 40^\circ 35'$, and $ABN = 95^\circ 07'$, were read. At A, the angle of elevation of M was $17^\circ 19'$, and the angle of elevation of N was $18^\circ 45'$. Compute the difference in the height of the two peaks.

Ans. M is 226.59 feet above N.

8. Draw a figure, and deduce a general rule for the double longitude of any course in terms of the double longitude and departure of the preceding course.

9. Explain what is meant by balancing a survey.

10. Describe fully how the latitudes and departures of a series of courses may be balanced, and how to determine the balanced length and bearing of each course.

REVIEW QUESTIONS

ON THE SUBJECT OF

PLANE SURVEYING

PART III

---

1. Let it be required to prepare a table of declinations for July 16, 1904, for a point whose latitude is 38° 30′, and which lies in the "Eastern Time" belt. The sun's apparent declination at Greenwich Mean Noon for that date is 21° 24.3′ and the hourly change is—24″ .38.

2. Explain the term "adjusting the triangle" and why adjustment is necessary.

3. A certain grade line has a fall of 12.5 feet in one-half of a mile. Determine the percentage of grade and the vertical angle corresponding to it.

4. In Fig. 103 given $BC=385$ feet; $ABC = 70° 05'$; $ACB=63° 28'$. Calculate the length of BD and the length of AD, and plot the figure accurately.

5. It is required to determine the linear convergence for a township situated in latitude 43° 18′ north.

6. In your own words describe fully the adjustments of the plane-table in their order.

7. Explain under what circumstances the transit and stadia may be used and when the plane-table may be used. Enumerate the advantages and disadvantages of each instrument.

8. In Fig. 95 the scale reading was $3\frac{1}{2}$ and the reading of the head 26. Determine the grade of the line AB.

9. A line is to be run at a grade of 4.75 per cent. Explain fully how this would be done with the gradienter.

10. In your own words describe the plane-table and its uses; its advantages and disadvantages.

11. Explain fully the organization of a topographic party using the transit and stadia and explain the method of keeping the

REVIEW QUESTIONS

ON THE SUBJECT OF

PLOTTING AND TOPOGRAPHY

---

1. What scale of map and what general method of field work would you adopt for surveying and plotting the smaller details of topography? Give reasons.

2. Define a contour. What is a contour interval and what conditions determine it?

3. Describe "hachures" and the principles on which they are drawn.

4. How would you determine contours on a small area if only a tape, rod, and hand-level were available? Explain why.

5. The elevations of two adjacent points are 68.2 and 87.9 feet, respectively; the points are 90 feet apart horizontally; locate the five-foot contours between them.

6. Vertical angles taken to the 6-foot and 1-foot marks on a rod held vertically gave the angles $-3° 23'$ and $-4° 07'$, respectively;

   (a) what is the true horizontal distance from instrument to rod?

   (b) what is the relative elevation of the ground at the rod and the telescope?

   (c) how much would the horizontal distance be altered by a change of one-half minute in either vertical angle?

7. What are the disadvantages of the gradienter which do not apply to the stadia method?

8. What are the limitations of the stadia method?

9. What are the three shortened methods of "reducing" stadia observations?

10. Describe the method of making a "simple conic projection." What portion of the resulting map is correctly repre-

REVIEW QUESTIONS

ON THE SUBJECT OF

HIGHWAY CONSTRUCTION

PART I

1. What are the essentials necessary for successful construction of bituminous coverings?
2. What data is necessary to design intelligently the size of culverts?
3. Discuss the preparation of the foundation of a road.
4. What is meant by *resistance to traction* and *resistance to rolling?*
5. What are the essentials of good maintenance and repair of broken-stone roads?
6. Discuss methods of dust suppression.
7. Discuss the office of wearing surface.
8. In making a reconnoissance for a road, what points should be considered?
9. Discuss briefly the making of gravel roads.
10. How are sand roads and clay roads improved?
11. On what conditions does the location of a road depend?
12. Describe the "mixing method" of applying bituminous covering.
13. How are broken-stone roads constructed?
14. What are the principal methods used in construction of concrete pavement?
15. What are the elements entering into the choice of final selection of roads?
16. Discuss methods of compacting broken stone.

REVIEW QUESTIONS

ON THE SUBJECT OF

HIGHWAY CONSTRUCTION

PART II

---

1. What are the advantages of granite-block pavement?
2. Draw a section showing proper construction of stone-block pavement.
3. What are the essentials of a good foundation?
4. What are the important conditions to bear in mind in fixing grades?
5. What substitutes may be used for pavement in country districts where funds are insufficient for constructing street pavements?
6. What are the disturbing forces that the curb has to resist?
7. What should comprise the essentials of a set of specifications?
8. What qualities are essential to a good pavement?
9. What are the essential points in a contract?
10. What are the materials generally used for foot paths?
11. Give the methods used for making roads passable during the period of snow fall.
12. What are the advantages claimed for asphalt blocks in comparison with a continuous sheet of asphalt?
13. Discuss the laying of asphalt pavement.
14. What governs the waterproof qualities of stone-block pavements?
15. What are the advantages of asphalt pavement?
16. How is the drainage of streets taken care of?

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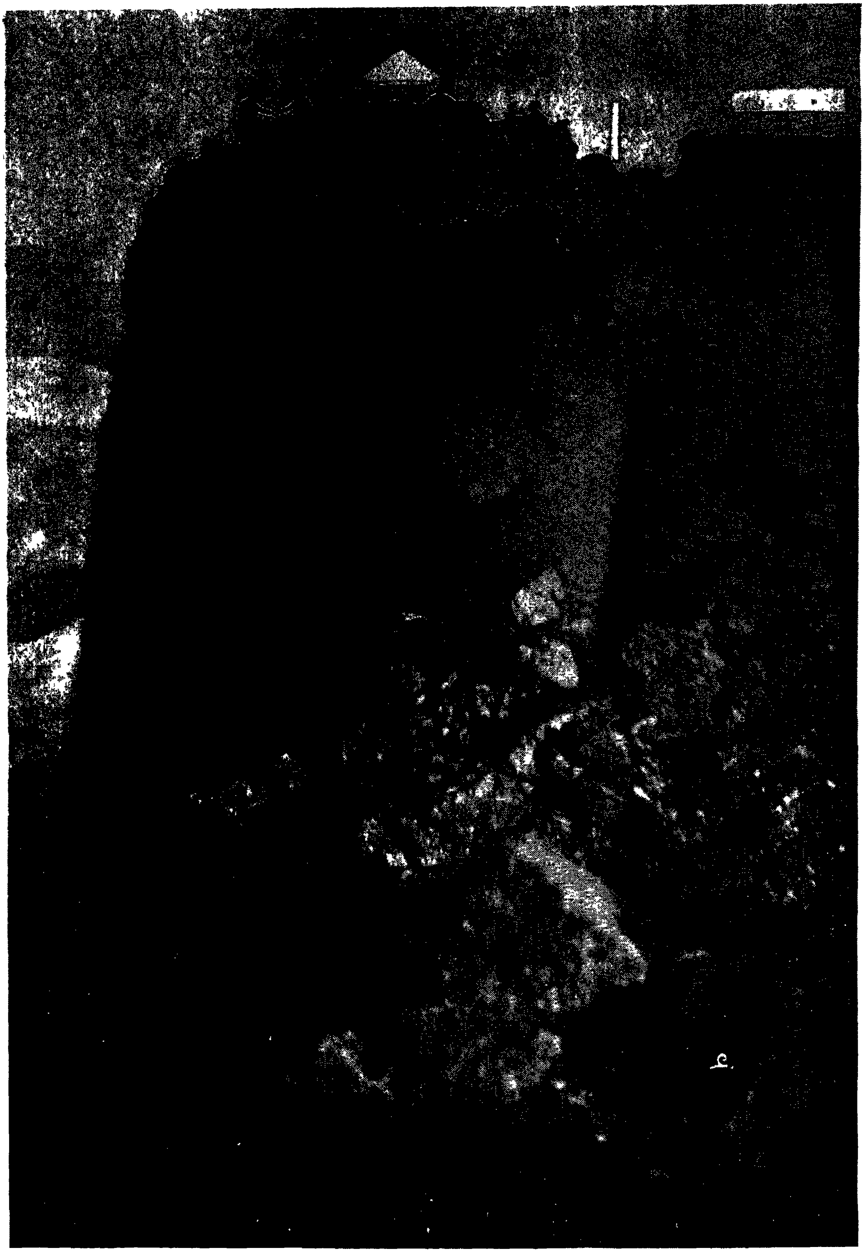

TRIANGULATION STATION ON MOUNT UNCOMPAHGRE. ELEVATION 14,300 FEET

Courtesy of U S Coast and Geodetic Survey, Washington, D C.

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